# Chapter 5
# Construction of Tunnels and Shafts

## 5-1. General

*a*. The design team must be composed of design and construction engineers and geologists experienced in underground construction. Methods and sequences of excavation affect the loads and displacements that must be resisted by initial and permanent ground support. The basic shape of an excavated opening must be selected for practicality of construction. Although it is good practice to leave many details of construction for the contractor to decide, it is often necessary for the designer to specify methods of construction when the choice of methods affects the quality or safety of the work or when construction will have environmental effects. There are aspects of construction where the design team may have to work closely with the contractor or include restrictive provisions in the specifications.

*b*. The basic components of underground construction include the following:

- Excavation, by blasting or by mechanical means.
- Initial ground support.
- Final ground support.

*c*. In the past, the terms “primary” and “secondary” support have been used for “initial” and “final” support. This usage is discouraged because it is misleading since in terms of end function, the final support has the primary role, and initial ground support is often considered temporary. However, in many instances today, initial ground support may also serve a function in the permanent support.

*d*. Other important components of construction include the following:

- Site and portal preparation.
- Surveying.
- Ventilation of the underground works.
- Drainage and water control.
- Hazard prevention.
- Controlling environmental effects.

These topics are discussed in this chapter; however, it is not the intent to present a complete guide to tunnel construction. The designer may have reason to explore in greater depth certain details of construction, such as blasting effects or TBM feasibility or projected advance rates.

## 5-2. Tunnel Excavation by Drilling and Blasting

While TBMs are used in many tunneling projects, most underground excavation in rock is still performed using blasting techniques. The design team should specify or approve the proposed method of excavation.

*a*. *The excavation cycle*. The typical cycle of excavation by blasting is performed in the following steps:

(1) Drilling blast holes and loading them with explosives.

(2) Detonating the blast, followed by ventilation to remove blast fumes.

(3) Removal of the blasted rock (mucking).

(4) Scaling crown and walls to remove loosened pieces of rock.

(5) Installing initial ground support.

(6) Advancing rail, ventilation, and utilities.

*b*. *Full- and partial-face advance*.

(1) Most tunnels are advanced using full-face excavation. The entire tunnel face is drilled and blasted in one round. Blastholes are usually drilled to a depth somewhat shorter than the dimension of the opening, and the blast “pulls” a round a little shorter (about 90 percent with good blasting practice) than the length of the blastholes. The depth pulled by typical rounds are 2 to 4 m (7-13 ft) in depth. Partial-face blasting is sometimes more practical or may be required by ground conditions or equipment limitations. The most common method of partial-face blasting is the heading-and-bench method, where the top part of the tunnel is blasted first, at full width, followed by blasting of the remaining bench. The bench can be excavated using horizontal holes or using vertical holes similar to quarry blasting. There are many other variations of partial-face blasting, such as a center crown drift, followed by two crown side drifts, then by the bench in one, two, or three stages.

(2) Reasons for choosing partial as opposed to full-face blasting include the following:

(a) The cross section is too large for one drill jumbo for example: Underground openings of the sizes usually required for powerhouses, valve chambers, and two- or three-lane highway tunnels are usually excavated using partial-face blasting excavation.

(b) The size of blast in terms of weight of explosives must be limited for vibration control.

(c) The ground is so poor that the full width of excavation may not be stable long enough to permit installation of initial ground support.

*c. Design of a blasting round.*

(1) The individual blasting rounds are usually designed by a blasting specialist in the contractor's employ. The design is reviewed by the engineer for compliance with specifications. Information about the detailed design of blasting rounds can be found, for example, in Langefors and Kihlstrom (1978) or Persson, Holmberg, and Lee (1993). Information about blasting agents and blasting design can also be found in handbooks published by explosives manufacturers, such as Blaster's Handbook (Dupont). See also EM 1110-2-3800. Blastholes are usually drilled using hydraulic percussion drills. The efficiency and speed of hole drilling has been improving rapidly, and bit wear and precision of drilling have also improved due to new designs of drill rods and bits. Drilling for small tunnels is often done with a single drill, but more often drill jumbos are used with two or more drills mounted. The jumbos can be rail, tire, or track mounted. Track-mounted straddle jumbos permit mucking equipment to move through the jumbo to and from the face.

(2) Effective blasting design requires attention to the degree of confinement for the detonation of each blast hole. If a blast hole is fully confined, the detonation may result merely in plastic deformation. With a nearby free face, the blast wave will create fractures toward the face, fragment the rock between the hole and the face, and remove the fragments. The distance to the free face, the burden, is taken generally between 0.75 and 1.0 times the hole spacing.

(3) In a tunnel, there is initially no free face parallel to the blasthole. One must be created by the blast design and this is done in one of several ways.

(a) The V-cut or fan-cut uses a number of holes drilled at an angle toward each other, usually in the lower middle of the face, to form a wedge. Detonation of these holes first will remove the material in the wedge and allow subsequent detonations to break to a free face.

(b) The burn cut uses parallel holes, most often four holes close together with only two loaded, or one or two large-diameter holes, usually up to 125 mm (5 in.) in diameter, unloaded. Remaining holes are laid out and initiated so that each new detonation in one or more blastholes always will break to a free face. The holes set off just after the cut are the stopping holes, also called easer, relief (reliever), or enlarger holes. The last holes to be detonated are the contour or trim holes around the periphery. The ones in the invert are called lifters.

(c) Perimeter holes are usually drilled with a lookout, diverging from the theoretical wall line by up to about 100 mm (4 in.) since it is not possible to drill right at the edge of the excavated opening. The size of the drill equipment requires a setback at an angle to cover the volume to be excavated. Successive blasts result in a tunnel wall surface shaped in a zigzag. Therefore, overbreak is generally unavoidable.

(d) Delays, electric or nonelectric, are used to control the sequence and timing of the detonations and to limit the amount of explosives detonated at any time. These are of several types. Millisecond delays are fast, ranging from 25 to 500 ms; other delays are slower. Up to 24 ms delays are available. Delays must be selected such that the rock fragments are out of the way before the next detonation occurs. Millisecond delays are often used within the burn part of the blast, with half-second delays used for the remainder. In the past, the blast was usually initiated electrically, using electrical blasting caps or initiators. Nonelectrical blast initiators and delays are now available and are often preferred because they are not affected by stray electric currents.

(e) Blasting agents are available for special purposes. They vary in charge density per length of hole, diameter, velocity of detonation, fume characteristics, water resistance, and other characteristics. In dry rock, the inexpensive ANFO (a mixture of ammonium nitrate and fuel oil) is often used. Trim holes require special blasting agents with a very low charge per meter. Blastholes are typically 45 to 51 mm (1.9-2 in.) in diameter. Sticks or sausages of explosive agents are usually 40 mm (1.6 in.) in diameter and are tamped in place to fill the hole, while those used for trim holes are often 25 mm (1 in.) in diameter and are used with stemming.

(f) Two parameters are often calculated from a blast design: the powder factor or specific charge (kilograms of explosives per cubic meter of blasted rock) and the drill factor (total length of drill holes per volume of blasted rock (meter/cubic meter)). These are indicators of the overall economy of blasting and permit easy comparison among blast patterns. The powder factor varies greatly with the conditions. It is greater when the confinement is greater, the tunnel smaller, or when the rock is harder and more resilient. Rocks with voids sometimes require large powder factors. For most typical tunnel blasting, the powder factor varies between 0.6 and 5 kg/m$^3$. The powder factor can vary from 1 kg/m$^3$ in a tunnel with an opening size greater than 30 m$^2$ to more than 3 kg/m$^3$ for a size less than 10 m$^2$, in the same type of ground. Typical drill factors vary between 0.8 and 6 m/m$^3$. Figure 5-1 shows a typical, well-designed round. This 19.5-m$^2$ round uses 40 holes with a powder factor of 1.9 kg/m$^3$ and a drill factor of 2.2 m/m$^3$. Typical powder factors and drill hole requirements are shown on Figures 5-2 and 5-3.

*d. Controlled blasting.*

(1) The ideal blast results in a minimum of damage to the rock that remains and a minimum of overbreak. This is achieved by controlled blasting. Control of rock damage and overbreak is advantageous for many reasons:

(a) Less rock damage means greater stability and less ground support required.

(b) The tunneling operations will also be safer since less scaling is required.

(c) Less overbreak makes a smoother hydraulic surface for an unlined tunnel.

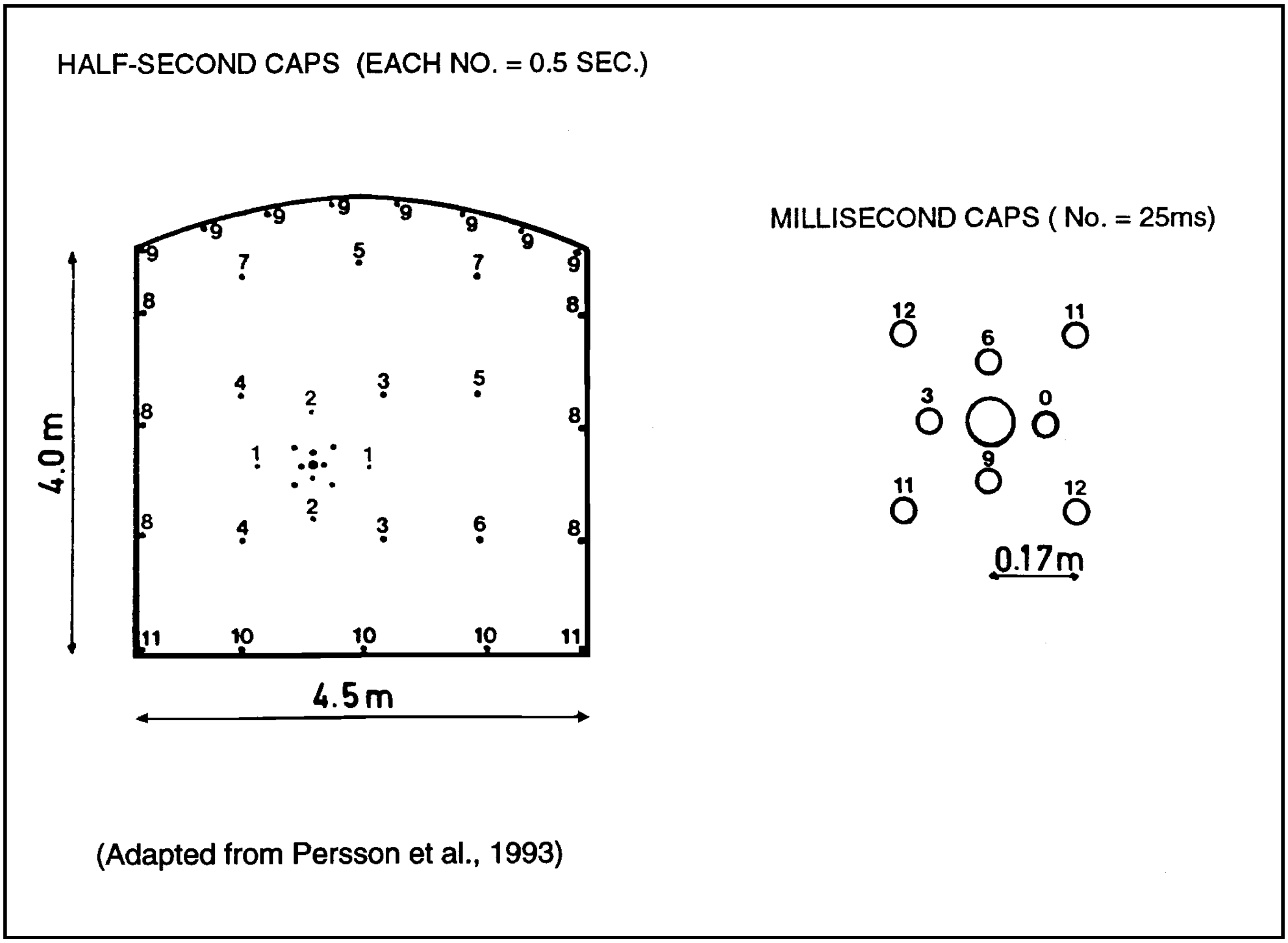



**Figure 5-1. Blasting round with burn cut blastholes 3.2 m, advance 3.0 m**

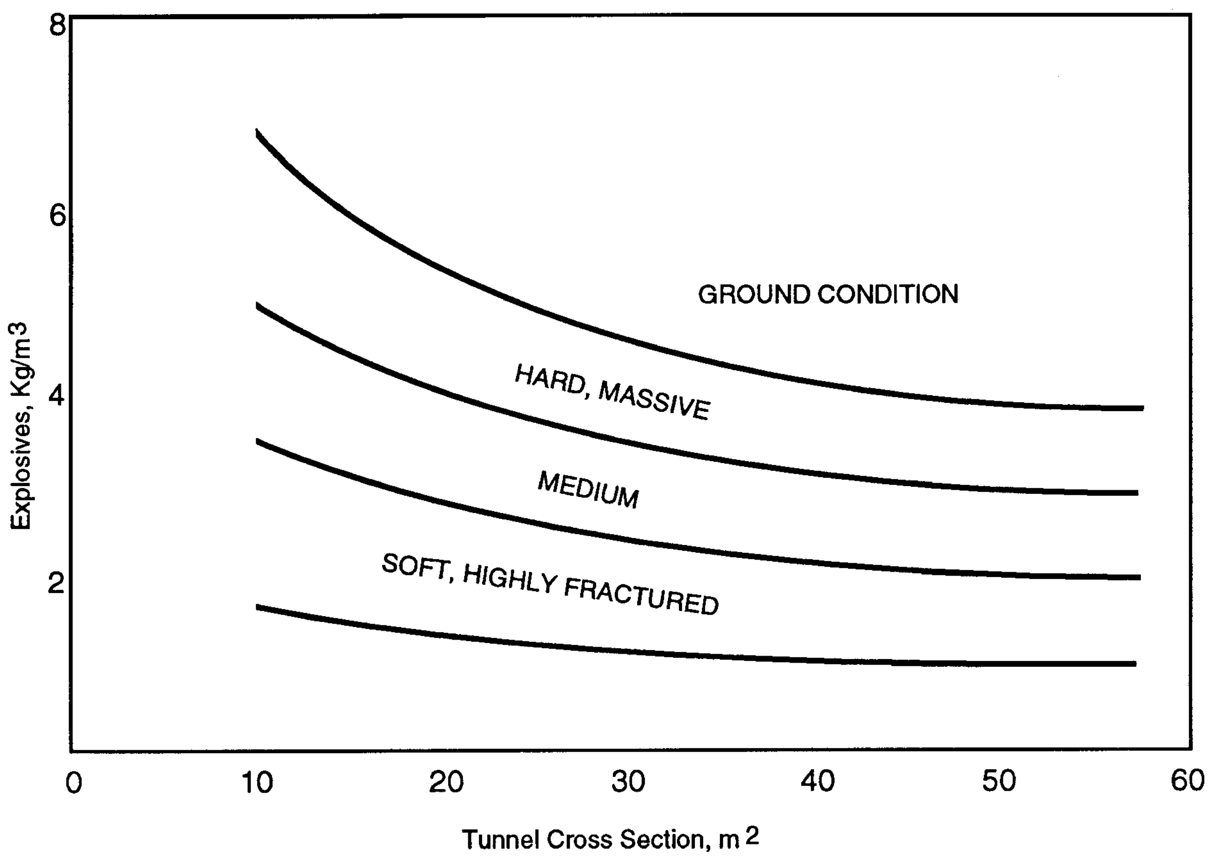



**Figure 5-2. Typical powder factors**

(d) For a lined tunnel, less overbreak means less concrete to fill the excess voids.

(2) Controlled blasting involves a closer spacing of the contour or trim holes, which are loaded lighter than the remainder of the holes. A rule often used is to space contour holes about 12-15 times hole diameter in competent rock, and 6-8 times hole diameter in poor, fractured rock. Because controlled blasting generally requires more blast holes than otherwise might be required, it takes longer to execute and uses more drill steel. For these reasons, contractors are often reluctant to employ the principles of controlled blasting.

(3) But controlled blasting requires more than just the design of proper perimeter blasting. Blast damage can occur long before the trim holes are detonated. Controlled blasting requires careful design and selection of all aspects of the round-geometry, hole diameter, hole charges, hole spacings and burdens, and delays—as well as careful execution of the work.

(4) One of the keys to successful controlled blasting is precise drilling of blast holes. Deviations of blastholes from their design locations quickly lead to altered spacings and burdens, causing blast damage and irregular surfaces. Modern hydraulic drills are not only quick but also permit better precision than was the norm. The highest precision is obtained with the use of computer-controlled drill jumbos in homogeneous rock.

(5) Inspection of the blasted surfaces after the blast can give good clues to the accuracy of drilling and the effectiveness with which blasting control is achieved. A measure of success is the half-cast factor. This is the ratio of half casts of blast holes visible on the blasted surface to

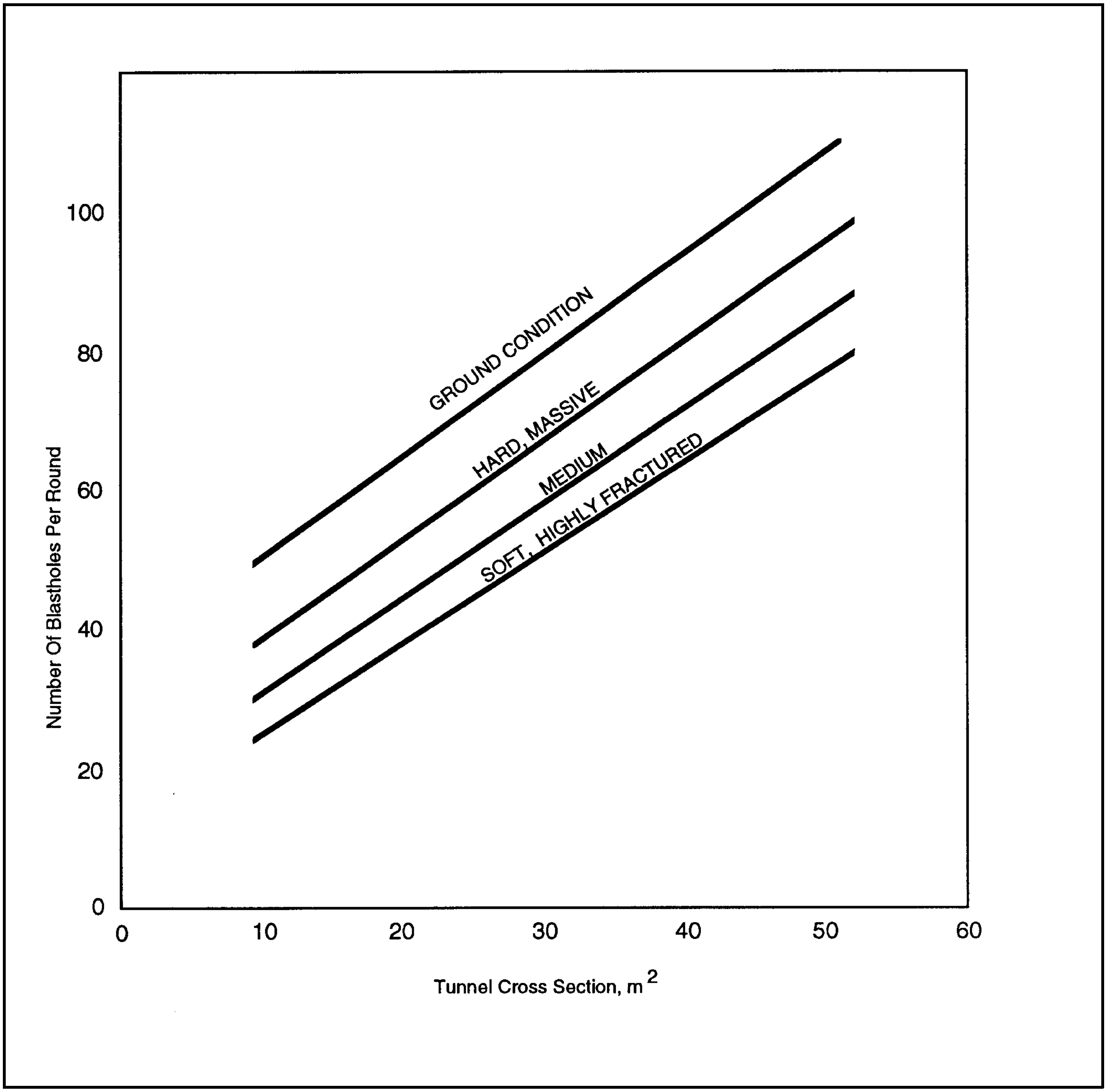



**Figure 5-3. Typical drill hole requirements**

the total length of trim holes. Depending on the quality of the rock and the inclination of bedding or jointing, a half-cast factor of 50 to 80 percent can usually be achieved. Irregularities in the surface caused by imprecise drilling are also readily visible and measurable. The regularity and appropriateness of the lookout should also be verified. Other means to verify the quality of blasting include methods to assess the depth of blast damage behind the wall. This may be done using seismic refraction techniques and borescope or permeability measurements in cored boreholes. The depth of the disturbed zone can vary from as little as 0.1-0.2 m (4-8 in.) with excellent controlled blasting to more than 2 m (7 ft) with uncontrolled blasting.

*e. Blast vibrations*. Blasting sets off vibrations that propagate through the ground as displacement or stress waves. If sufficiently intense, these waves can cause damage or be objectionable to the public. Vibration control is particularly important in urban environments. Monitoring and control of blasting are described in detail in several publications, including Dowding (1985).

(1) The intensity of blasting vibrations felt a given distance from the blast is a function of the following factors:

- The total charge set off by each individual delay (a delay as small as 8 ms is sufficient to separate two detonations so that their blast wave effects do not overlap).
- The distance from the detonation to the point of interest.
- The character of the ground (high-modulus rock permits the passage of waves of higher frequency, which quickly damp out in soil-like materials).
- The degree of confinement of the blast (the greater the confinement, the greater percentage of the total energy will enter the ground as vibration energy).
- Geometric site features will sometimes focus the vibration energy, as will geologic features such as bedding with hard and soft layers.

With a given explosive charge and a given distance, the intensity of vibration can be estimated using scaling laws. Most commonly, the square-root scaling law is used, which says that the intensity of the vibration is a function of the square root of the charge, *W*. The most important vibration parameter is the peak particle velocity, *V*.

$$V = H\ (D/W^{1/2})^{-B}$$

where *B* is an empirically determined power. The quantity $D/W^{1/2}$ is called the scaled distance, and *H* is the peak velocity at a scaled distance of one. This relationship plots as a straight line on a log-log plot of velocity against scaled distance, with *D* in meters, *W* in kilograms of explosive, *V* in millimeters/second. The quantity *H* varies with blast characteristics, confinement, and geologic environment. A typical range for *H* is 100 to 800 (metric); for a given geologic medium, *H* can vary within a single blast: 250 for the V-cut, 200 for production holes, and 150 for the trim holes. *H* is generally smaller for shorter rounds. The power *B* can vary from 0.75 to 1.75; it is often taken as 1.60. For a particular site or environment, the empirical relationship can be developed based on trial blasts, using the log-log plot. Many factors affect the measured vibrations and a precise relationship is not likely to evolve. Rather, ranges of data are used to develop a safe envelope for production blasting. Typical ranges of peak particle velocity as a function of scaled distance are shown on Figure 5-4. A typical relationship between allowable charge per delay and distance for a vibration limit of 50 mm/s (2 in./s) is shown in Table 5-1 (SME 1992).

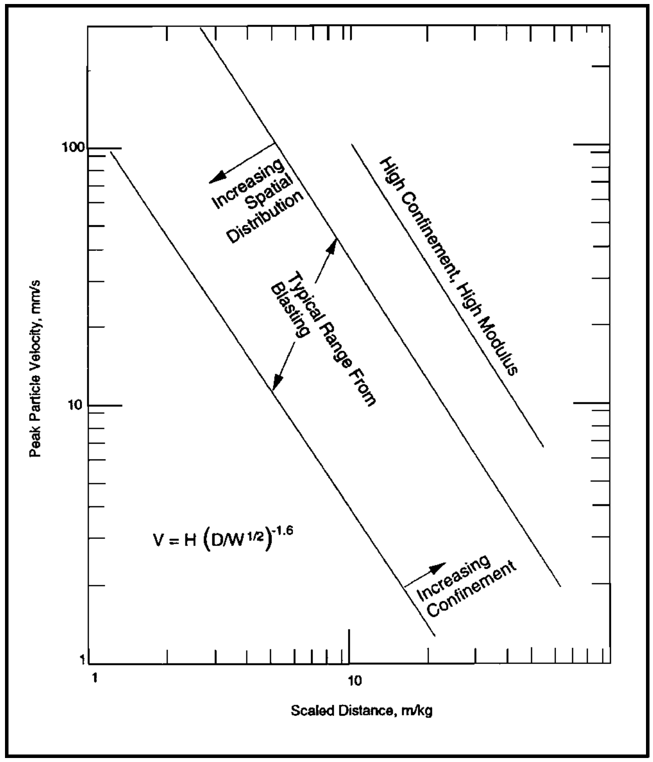



**Figure 5-4. Ground vibrations from blasting**

**Table 5-1**
**Allowable Change per Delay**

| Allowable Charge, lb | Distance, ft |
|---|---|
| 0.25 | 10 |
| 1.0 | 20 |
| 6 | 50 |
| 25 | 100 |
| 156 | 250 |

(2) Damage to structures caused by blasting is related to peak particle velocity. It is generally recognized that a peak particle velocity of 50 mm/s (2 in./s) will not damage residential structures or other buildings and facilities. In fact, most well-built structures can withstand particle

velocities far greater than 50 mm/s (2 in./s); however, it is the generally accepted limit for blasting vibrations.

(3) When blasting is carried out in the vicinity of fresh concrete, peak velocities must be restricted to avoid damage to the concrete. This concern is discussed in some detail in the Underground Mining Methods Handbook (SME 1992). Both structural concrete and mass concrete are relatively insensitive to damage when cured. Concrete over 10 days old can withstand particle velocities up to 250 mm/s (10 in./s) or more. Very fresh concrete that has not set can withstand 50 mm/s (2 in./s) or more. On the other hand, young concrete that has set is subject to damage. The peak particle velocity in this case may have to be controlled to under 6 mm/s (0.25 in./s), and particle velocities should not exceed 50 mm/s (2 in./s) until the concrete is at least 3 days old. These values may vary with the character of the foundation rock, the setting time and strength of the concrete, the geometry of the structure, and other characteristics. For important structures, site-specific analysis should be conducted to set blasting limits.

(4) Damage of intact rock in the form of microfractures usually does not occur below particle velocities of 500-1,000 mm/s, depending on the strength of the rock.

(5) Human perception is far more sensitive to blasting vibrations than are structures. Vibrations are clearly noticeable at peak particle velocities as low as 5 mm/s (0.2 in./s) and disturbing at a velocity of 20 mm/s (0.8 in./s). Perception of vibrations is, to a degree, a function of the frequency of the vibrations; low-frequency vibrations (<10-15 Hz) are more readily felt than high-frequency vibrations. Furthermore, vibrations may be much more objectionable during night hours. Setting acceptable blasting limits in an urban area requires adherence to locally established codes and practice. If codes do not exist, public participation may be required in setting peak velocity limits. The U.S. Bureau of Mines (Siskind et al. 1980) has made recommendations on peak particle velocities as shown on Figure 5-5 that may be used when no local ordinances apply. Two examples of blasting limits in urban areas follow:

(a) For construction of the TARP system in Chicago, blasting was limited to the hours of 8 a.m. and 6 p.m. Peak particle velocity at inhabited locations were limited to 12.5 mm/s (0.5 in./s) for the frequency range of 2.6-40 Hz; 18.75 mm/s (0.75 in./s) for the range above 40 Hz, and lower than 12.5 mm/s (0.5 in./s) for frequencies under 2.6 Hz. These kinds of restrictions resulted in contractors generally choosing mechanical excavation methods rather than blasting for shafts.

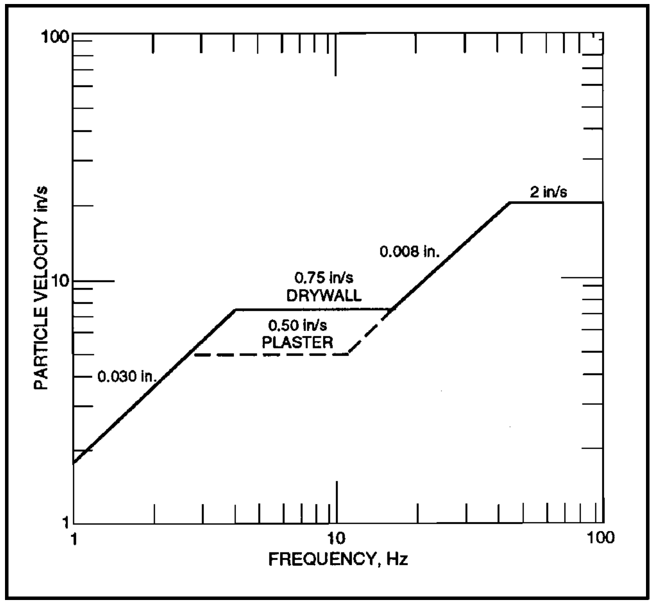



**Figure 5-5. U.S. Bureau of Mines recommended blasting level criteria**

(b) For MARTA construction in Atlanta, peak velocity was restricted to 25 mm/s (1 in./s) at the nearest inhabited structure and 50 mm/s (2 in./s) at the nearest uninhabited structure. Between 10 p.m. and 7 a.m., velocities were limited to 15 mm/s (0.6 in./s). Air blast overpressures were also restricted.

*f. Mucking.*

(1) Muck removal requires loading and conveying equipment, which can be trackless (rubber tired, in shorter tunnels) or tracked (rail cars, in longer tunnels) or belt conveyors. Provisions for passing trains or vehicles must be provided in long tunnels. Because of the cyclic nature of blasting excavation, great efficiency can be achieved if crews and equipment can work two or more tunnel faces at the same time.

(2) The ideal blast results in breaking the rock such that few pieces are too large to handle; however, excessive fines usually mean waste of explosive energy. The muck pile can be controlled by the timing of the lifter hole detonation. If they are set off before the crown trim holes, the pile will be compact and close to the face; if they are set off last, the pile will be spread out, permitting equipment to move in over the muck pile.

*g. Scaling.* An important element of excavation by blasting is the scaling process. Blasting usually leaves behind slivers or chunks of rock, loosened and isolated by blast fractures but remaining tenuously in place. Such chunks can fall after a period of some time, posing significant danger to personnel. Loose rock left in place can also result in nonuniform loads on the permanent lining. Loosened rock is usually removed by miners using a heavy scaling bar. This work can be dangerous and must be conducted with great care by experienced miners. Tools are now available to make this a much less dangerous endeavor. Hydraulically operated rams or rock breakers can be mounted at the end of a remotely operated hydraulic arm. This greatly reduces the hazard and may improve the speed with which this task is accomplished.

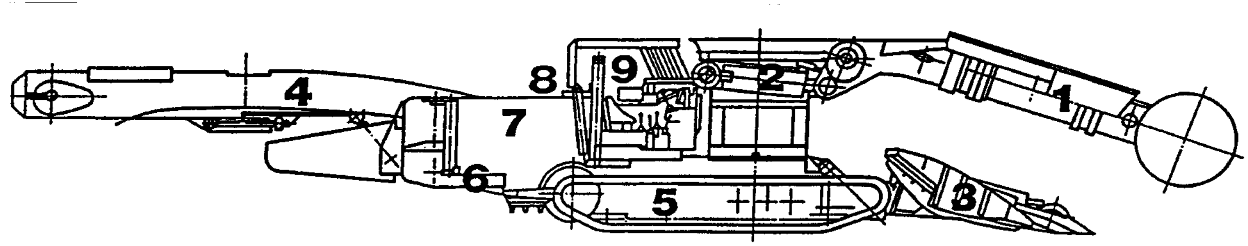
## 5-3. Tunnel Excavation by Mechanical Means

Much underground excavation today is performed by mechanical means. Tools for excavation range from excavators equipped with ripper teeth, hydraulic rams, and roadheaders to TBMs of various designs. By far, TBMs are the most popular method of excavation. Roadheaders are versatile machines, useful in many instances where a TBM is not cost-effective. This section describes roadheader and TBM excavation methods and the factors that affect the selection of mechanical excavation methods.

*a. Roadheader excavation.* Roadheaders come in many sizes and shapes, equipped for a variety of different purposes. They are used to excavate tunnels by the full-face or the partial-face method, and for excavation of small and large underground chambers. They may also be used for TBM starter tunnels, ancillary adits, shafts, and other underground openings of virtually any shape and size, depending on rock hardness limitations. Most roadheaders include the following components:

- Rotary cutterhead equipped with picks.
- Hydraulically operated boom that can place the cutterhead at a range of vertical locations.
- Turret permitting a range of horizontal motion of the cutterhead.
- Loading device, usually an apron equipped with gathering arms.
- Chain or belt conveyor to carry muck from the loading device to the rear of the machine for off-loading onto a muck car or other device.
- Base frame, sometimes with outriggers or jacks for stabilization, furnished with electric and hydraulic controls of the devices and an operator's cab.
- Propelling device, usually a crawler track assembly.

A typical, large roadheader is shown on Figure 5-6.




1. Cutter Boom
2. Turret
3. Loading Device
4. Chain Conveyor
5. Crawler Track Assembly
6. Base Frame
7. Electrics
8. Hydraulics
9. Operator's Cab

**Figure 5-6. Alpine Miner 100**

(1) Several types and sizes of cutterheads exist. Some rotate in an axial direction, much like a dentists drill, and cut the rock by milling as the boom forces the cutterhead, first into the face of the tunnel, then slewing horizontally or in an arch across the face. Others rotate on an axis perpendicular to the boom. The cutterhead is symmetrical about the boom axis and cuts the rock as the boom moves up and down or sideways. The cutterhead is equipped with carbide-tipped picks. Large radial drag picks or forward attack picks are used, but the most common are the point attack picks that rotate in their housings. The spacing and arrangement of the picks on the cutterhead can be varied to suit the rock conditions and may be equipped with high-pressure water jets in front of or behind each pick, to cool the pick, improve cutting, remove cuttings, and suppress dust generation. Depending on the length of the boom and the limits of the slewing and elevating gear, the cutterhead can reach a face area of roughly rectangular or oval shape. The largest roadheaders can cut a face larger than 60 $m^2$ from one position. Booms can be extended to reach further, or can be articulated to excavate below the floor level, or may be mounted on different bodies for special purposes, such as for shaft excavation, where space is limited.

(2) Most roadheaders can cut rock with an unconfined compressive strength of 60 to 100 MPa (10,000-15,000 psi). The most powerful can cut rock with a strength of 150 MPa (22,000 psi) to 200 MPa (30,000 psi) for a limited duration. Generally, roadheaders cut most effectively into rocks of a strength less than 30 MPa (5,000 psi), unless the rock mass is fractured and bedded. The cutting ability depends to a large measure on the pick force, which again depends on the torque available to turn the cutterhead, the cutterhead thrust, slewing, and elevating forces. The advance rate depends on the penetration per cut and the rotary speed of the cutterhead. The torque and speed of the cutterhead determines the power of the head. Cutting hard rock can be dynamic and cause vibrations and bouncing of the equipment, contributing to component wear; therefore, a heavy, sturdy machine is required for cutting hard rock. Typical small-to-medium roadheaders weigh about 20 to 80 tons and have available cutterhead power of 30 to 100 KW, total power about 80 to 650 KW. The larger machines weigh in excess of 90 tons, with cutterhead power of up to 225 KW. With a well-stabilized roadheader body, a cutterhead thrust of more than 50 tons can be obtained.

(3) Roadheader performance in terms of excavation rate and pick consumption can be predicted based on laboratory tests. Types of tests and examinations typically performed include thin-section analysis to determine the cementation coefficient and the quartz content and Shore scleroscope and Schmidt hammer hardness tests. Density, porosity, compressive, and tensile strength tests are also useful. Bedding and jointing also affect the efficiency of cutting. In a heavily jointed mass, ripping and loosening of the jointed mass can be more important than cutting of the intact rock. Bedding planes often facilitate the breaking of the rock, depending on the direction of cutterhead rotation relative to the bedding geometry. An experienced operator can take advantage of the observed bedding and jointing patterns to reduce the energy required to loosen and break the rock, by properly selecting the pattern and sequence of excavating the face. The selection of equipment should be made without regard to the potential benefits from the bedding and jointing. The equipment should be capable of cutting the intact rock, regardless of bedding and jointing.

*b. Excavation by tunnel boring machine*. A TBM is a complex set of equipment assembled to excavate a tunnel. The TBM includes the cutterhead, with cutting tools and muck buckets; systems to supply power, cutterhead rotation, and thrust; a bracing system for the TBM during mining; equipment for ground support installation; shielding to protect workers; and a steering system. Back-up equipment systems provide muck transport, personnel and material conveyance, ventilation, and utilities.

(1) The advantages of using a TBM include the following:

- Higher advance rates.
- Continuous operations.
- Less rock damage.
- Less support requirements.
- Uniform muck characteristics.
- Greater worker safety.
- Potential for remote, automated operation.

(2) Disadvantages of a TMB are the fixed circular geometry, limited flexibility in response to extremes of geologic conditions, longer mobilization time, and higher capital costs.

(3) A database covering 630 TBM projects from 1963 to 1994 has been assembled at The University of Texas at Austin (UT). This database supplies information on the

range of conditions and performance achievements by TBMs and includes 231 projects from North America, 347 projects from Europe, and 52 projects from other locations. A brief summary of the database is presented in Table 5-2. In addition, this database includes information on site geology and major impacts on construction. These are summarized in Table 5-3. Most database projects were excavated in sedimentary rock, with compressive strength between 20 and 200 MPa.

**Table 5-2**
**Description of Projects in the UT Database**

| Description | | Number Among Database Projects |
|---|---|---|
| No. of Projects in Completion Date Interval (total 630 projects) | 1963-1970 | 26 |
| | 1971-1975 | 53 |
| | 1976-1980 | 122 |
| | 1981-1985 | 139 |
| | 1986-1990 | 176 |
| | 1991-1994 | 114 |
| Total Project Lengths, km (total tunnel length in data base = 2,390 km) | 1963-1970 | 81 |
| | 1971-1975 | 134 |
| | 1976-1980 | 400 |
| | 1981-1985 | 530 |
| | 1986-1990 | 666 |
| | 1991-1994 | 579 |
| No. Projects in Excavated Diameter Interval, m | 2 to 3.5 m | 219 |
| | 3.6 to 5.0 m | 237 |
| | 5.0 to 6.5 m | 104 |
| | 6.5 to 8.0 m | 36 |
| | >8.0 m | 34 |
| No. Projects in Shaft Depth Interval | No shafts | 402 |
| | <15 m | 35 |
| | 15 to 50 m | 92 |
| | >50 m | 101 |
| No. Projects in Gradient Interval | >+20% uphill | 40 |
| | +10 to +20% | 6 |
| | +3 to +10% | 1 |
| | + 3 to - 3 % | 573 |
| | -3 to -10% | 3 |
| | -10 to -20% | 7 |
| | >-20% down | 0 |
| No. TBMs in Indicated Starting Condition | New | 318 |
| | Direct Reuse | 22 |
| | Refurbished | 261 |
| | Unspecified | 29 |
| No. TBMs with Indicated Shield Types | Open | 512 |
| | Single Shield | 56 |
| | Double Shield | 38 |
| | Special Shield | 15 |
| | Unspecified | 9 |

**Table 5-3**
**Description of Rock and Problems Encountered on Projects in the UT Database**

| Descriptor | | Among Database Projects |
|---|---|---|
| Predominant Geology (% of projects) | Sedimentary | 60% |
| | Metamorphic | 30% |
| | Igneous | 10% |
| Uniaxial Compressive Strength, MPa [96 average] [3 - 300 range] | <20 MPa | 11% |
| | 20-80 MPa | 28% |
| | 80-200 MPa | 52% |
| | >200 MPa | 9% |
| Projects with Special Problems | | Number of Projects |
| Mucking capacity limitation | | 7 |
| Excessive cutter wear | | 18 |
| Gassy ground | | 25 |
| Wide range in rock strength | | 43 |
| Wide range in rock mass quality | | 108 |
| Wide range in both rock strength and rock mass quality | | 14 |
| High water inflow | | 23 |
| Soil/weathered material | | 14 |
| Major fracture zones | | 33 |
| Overstressed rock | | 7 |
| Major equipment breakdown | | 30 |
| Contract stoppage | | 9 |

*c. TBM system design and operation.* A TBM is a system that provides thrust, torque, rotational stability, muck transport, steering, ventilation, and ground support. In most cases, these functions can be accomplished continuously during each mining cycle. Figure 5-7 is a sketch of a typical open or unshielded TBM designed for operation in hard rock. The TBM cutterhead is rotated and thrust into the rock surface, causing the cutting disc tools to penetrate and break the rock at the tunnel face. Reaction to applied thrust and torque forces may be developed by anchoring with braces (grippers) extended to the tunnel wall, friction between the cutterhead/shield and the tunnel walls, or bracing against support installed behind the TBM.

*d. TBM performance parameters.*

(1) TBM system performance is evaluated using several parameters that must be defined clearly and used consistently for comparative applications.

(a) *Shift time.* Some contractors will use 24-hr shifting and maintain equipment as needed "on the fly." As

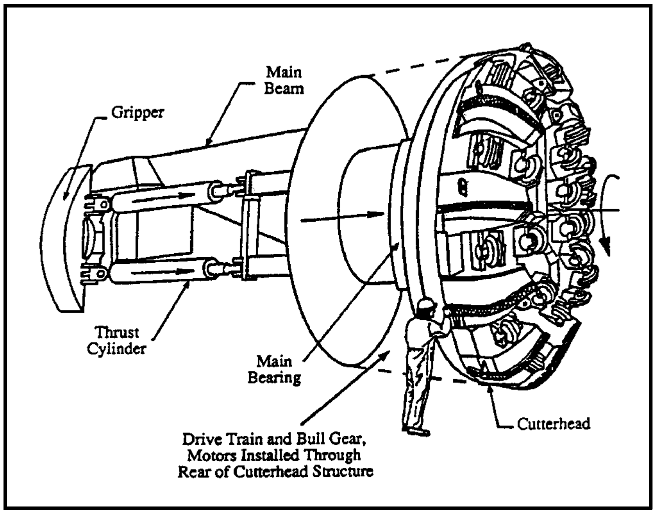



**Figure 5-7. Unshielded TBM schematic drawing**

used here, the shift time on a project is all working hours, including time set aside solely for maintenance purposes. All shift time on a project is therefore either mining time when the TBM operates or downtime when repairs and maintenance occur. Therefore,

$$\textit{Shift time} = \textit{TBM mining time} + \textit{Downtime}$$

(b) *Penetration rate*. When the TBM is operating, a clock on the TBM will record all operating time. The TBM clock is activated by some minimum level of propel pressure and/or by a minimum torque and the start of cutterhead rotation. This operating time is used to calculate the penetration rate (*PR*), as a measure of the cutterhead advance per unit mining time.

Therefore,

$$PR = \textit{distance mined}/\textit{TBM mining time}$$

*PR* is often calculated as an average hourly value over a specified basis of time (i.e., instantaneous, hour, shift, day, month, year, or the entire project), and the basis for calculation should be clearly defined. When averaged over an hour or a shift, *PR* values can be on the order of 2 to 10 m per hour. The *PR* can also be calculated on the basis of distance mined per cutterhead revolution and expressed as an instantaneous penetration or as averaged over each thrust cylinder cycle or other time period listed above. The particular case of penetration per cutterhead revolution is useful for the study of the mechanics of rock cutting and is here given the notation PRev (penetration per revolution). Typical values of PRev can be 2 to 15 mm per revolution.

(c) *Utilization*. The percentage of shift time during which mining occurs is the Utilization, *U*.

$$U\ (\%) = \textit{TBM mining time}/\textit{Shift Time} \times 100$$

and is usually evaluated as an average over a specified time period. It is particularly important that *U* is reported together with the basis for calculation—whole project (including start-up), after start-up "production" average, or *U* over some other subset of the job. On a shift basis, *U* varies from nearly 100 percent to zero. When evaluated on a whole project basis, values of 35 to 50 percent are typical. There is no clear evidence that projects using a reconditioned machine have a lower *U* than projects completed with a new machine. Utilization depends more on rock quality, equipment condition, commitment to maintenance, contractor capabilities, project conditions (entry/access, alignment curves, surface space constraints on operations), and human factors (remoteness, underground temperature, and environment).

(d) *Advance rate (AR)*. *AR* is defined on the basis of shift time as:

$$AR = \textit{Distance mined}/\textit{Shift time}$$

If *U* and *PR* are expressed on a common time basis, then the *AR* can be equated to:

$$AR = PR\ U\ (\%)/100$$

Advance rate can be varied by changes in either *PR* (such as encountering very hard rock or reduced torque capacity when TBM drive motors fail) or in *U* changes (such as encountering very poor rock, unstable invert causing train derailments, or highly abrasive rock that results in fast cutter wear).

(e) *Cutting rate (CR)*. CR is defined as the volume of intact rock excavated per unit TBM mining time. Again, the averaging time unit must be defined clearly, and typical values of CR range from 20 to 200 $m^3$ per TBM mining hour.

(2) TBM performance from the UT database is summarized in Table 5-4. Other performance parameters deal

**Table 5-4**
**TBM Performance Parameters for Projects in the UT Database**

| Parameter | Average | Range |
|---|---|---|
| Project length, km | 3.80 | 0.1 - 36.0 |
| Diameter, m | 4.4 | 2.0 - 12.2 |
| Advance rate, m/month | 375 | 5 - 2,084 |
| Advance rate, m/shift hr | 1.2 | 0.3 - 3.6 |
| Penetration rate m/TBM hr | 3.3 | 0.6 - 8.5 |
| Penetration rate mm/cutterhead revolution | 7.2 | 1.0 - 17.0 |
| Utilization, % | 38 | 5 - 69 |

with evaluation of disc cutter replacement rate, which depends on cutter position and type of cutter, rock properties and also thrust, diameter, and cutterhead rotation rate. Parameters used to evaluate cutter replacement rates include average TBM mining time before replacement, linear distance of tunnel excavated per cutter change, distance rolled by a disc cutter before replacement (the rolling life), and rates of material wear from disc measurements (expressed as weight loss or diameter decrease). Rolling life distances for the replaceable steel disc edges may be 200 to 400 km for abrasive rock, to more than 2,000 km for nonabrasive rock, and is longer for larger diameter cutters. Appendix C contains information on TBM performance evaluation and cost estimating.

*e. General considerations for TBM application.* Important project features that indicate use of TBM include low grades (<3 percent preferable for tunnel mucking and groundwater management) and driving up hill. A minimum grade of 0.2 percent is required for gravity drainage of water inflow. Horizontal curves in an alignment can be negotiated by an open TBM with precision and little delay if curve radii are on the order of 40 to 80 m. Most shielded TBMs and back-up systems are less flexible, however, so that a minimum radius of 150 to 400 m should generally be used for design purposes. Tighter curves should be avoided or planned in conjunction with a shaft to facilitate equipment positioning.

(1) Experience indicates that tunnel depth has little impact on advance rates in civil projects, providing that the contractor has installed adequate mucking capacity for no-delay operation. Therefore, tunnel depth should be chosen primarily by location of good rock. Portal access, as opposed to shafts, will facilitate mucking and material supply, but more important is that the staging area for either shaft or portal be adequate for contractor staging. Confined surface space can have a severe impact on project schedule and costs. For long tunnels, intermediate access points can be considered for ventilation and mucking exits. However, assuming the contractor has made appropriate plans for the project, a lack of intermediate access may not have a significant impact on project schedule.

(2) In planning a project schedule, the lead time needed to get a TBM onsite varies from perhaps 9 to 12 months for a new machine from the time of order, to perhaps 3 to 6 months for a refurbished machine, and to nearly no time required for a direct re-use without significant repairs or maintenance. With proper maintenance, used TBMs can be applied reliably, and there is little need to consider specifying new equipment for a particular project. TBM cutterheads can be redesigned to cut excavated diameters different by 1 to 2 m, but the thrust and torque systems should also be modified accordingly.

(3) With delivery of a TBM onsite, about 3 to 6 weeks will be required for assembly, during which time a starter tunnel should be completed. The start of mining rarely occurs with the full back-up system in place. Decreased advance rates on the order of 50 percent less than for production mining should be expected for the first 4 to 8 weeks of mining, as the back-up system is installed and the crew learns the ropes of system operation.

*f. Specification options for TBMs.* Specifications can be either prescriptive or performance specifications. If specifications include prescriptive information on performing work and also specific standards to be achieved in the finished product, disputes are likely. Make sure all specification provisions are compatible with provisions in the GDSR. If there are discrepancies or ambiguities, disputes can be expected.

(1) *New versus reconditioned equipment.* There is no statistically significant difference in performance between new and reconditioned equipment. Leaving the option open for contractors will tend to decrease costs. Exceptions include very long tunnels for which major equipment downtime for main-bearing repairs would be disastrous and hazardous ground conditions for which special TBM capabilities are required. Rebuilds are possible to ±10 percent of the original TBM diameter, but consideration should be given to the need to upgrade the thrust and torque systems if TBM diameter is increased, particularly if there is a significant difference in the rock between the previous and

current project. A given TBM may perform acceptably in weaker rock, but may be underpowered for harder rock.

(2) *Level of detail in specifications*. The key here is to specify only what is required by the designer for success in mining and support. Performance specifications are preferable. Reasonable specification requirements might include the following:

(a) Expected short stand-up time where support installation must be rapidly placed.

(b) Squeezing ground conditions with which the shield must be able to cope.

(c) Adequate groundwater handling system capacity onsite.

(d) Special equipment, safety management, and special operating procedures for gassy ground.

(e) Expectations for the contractor to supply a TBM capable of a minimum PRev, and a back-up system sized to provide no-delay mucking.

(3) *Contractor submittals*. The designer should ask for only what is important and what he or she is prepared to review. For example, an engineer could ask the contractor to demonstrate that the mucking system capacity will be adequate to support no-delay mining, or the contractor might be asked for information on time to install support if stand-up time is expected to be critical to the mining operation.

*g*. *Record keeping and construction monitoring*. During construction, it is very important that the resident staff gather information concerning the progress of construction and the encountered ground conditions. Such information is paramount to understand and document any changing ground conditions and to evaluate the impact of changing conditions on the operations of the contractor and vice versa. The information important to monitoring TBM construction include the following:

(1) Shift records of contractor activities should be maintained throughout a contract, but primarily at the heading. Shift reports should include the following information:

(a) Sequential time log of each shift including all activities.

(b) Downtime including reasons for shutdown.

(c) Record of thrust and torque (motor-operating amperage, number of motors on line, cutterhead rotation rate, thrust pressure, and gripper pressure), tunnel station, and TBM clock time elapsed for each stroke cycle of mining.

(d) Record of all cutters changed, including TBM clock time and station for each replacement, disc position on the cutterhead and reason for replacement (such as disc wear, bad bearing, split disc, etc.).

(e) Start and end station for each shift and for each stroke cycle.

(f) Information on ground conditions, groundwater encountered, and support installed, identified by station.

(g) Information on survey/alignment control and start/end of alignment curves.

(2) Records of installed support should be maintained in detail by the resident engineer. These can be incorporated in the tunnel geologic maps.

(3) Maps of as-encountered geologic conditions should be made for all tunnels driven with open TBMs. For shielded TBMs, all opportunities to view the rock at the heading should be mapped. The site geologist should maintain maps of the tunnel walls and changing ground conditions together with an assessment of rock mass quality and should continue to compare mapped information with predictions made at the time of site investigation and update or anticipate any notable systematic changes.

## 5-4. Initial Ground Support

*a*. *General*

Initial ground support is usually installed concurrently with the excavation. For drill and blast excavations, initial ground support is usually installed after the round is shot and mucked out and before drilling, loading, and blasting of the next round. For TBM-driven tunnels, excavation is carried out more or less continuously, with the support installed as the TBM moves forward. Because of the close relationship between excavation and initial support activities, they must be well coordinated and should be devised such that the process is cyclic and routine. Initial ground support may consist of steel ribs, lattice girders, shotcrete, rock dowels, steel mesh, and mine straps. The main purposes served by these support elements include stabilizing and preserving the tunnel after excavation and providing worker safety. As the quality of the rock increases, the amount of required initial ground support decreases. After installation of initial ground support, no other additional support may be required. In this case, the initial support will also fulfill the role of final support. In other cases, additional support, such as a cast-in-place concrete lining, may be installed. The initial and the final ground support then comprise a composite support system. An example of tunnel support fulfilling the initial and final support functions is when precast concrete segmental linings are used to support a tunnel in weak rock behind a TBM. One issue that must be considered when contemplating the use of initial support for final support is the longevity of the initial support components. While these components may behave satisfactorily in the short term, phenomena such as corrosion and deformation must be considered for permanent applications.

*b. Initial ground reinforcement*. Initial ground reinforcement consists of untensioned rock dowels and, occasionally, tensioned rock bolts. These are referred to as ground reinforcement, because their function is to help the rock mass support itself and mobilize the inherent strength of the rock as opposed to supporting the full load of the rock. It is much more economical to reinforce the rock mass than to support it. The reinforcement elements are installed inside the rock mass and become part of the rock mass. Rock support such as concrete linings and steel sets restrict the movements of the rock mass and offer external support to the rock mass. The design and construction of rock reinforcement systems are discussed in EM 1110-1-2907. The subject is addressed herein only as it relates to the construction of tunnels. There are three types of rock bolts (Stillborg 1986):

- Mechanically anchored (rock bolts) (Figure 5-8).
- Grouted bars (dowels) (Figures 5-9 and 5-10).
- Friction dowels (Figures 5-11 and 5-12).

Friction dowels are usually considered temporary reinforcement because their long-term corrosion resistance is uncertain. Typical technical data on these types of rock bolts and dowels are given in Table 5-5.

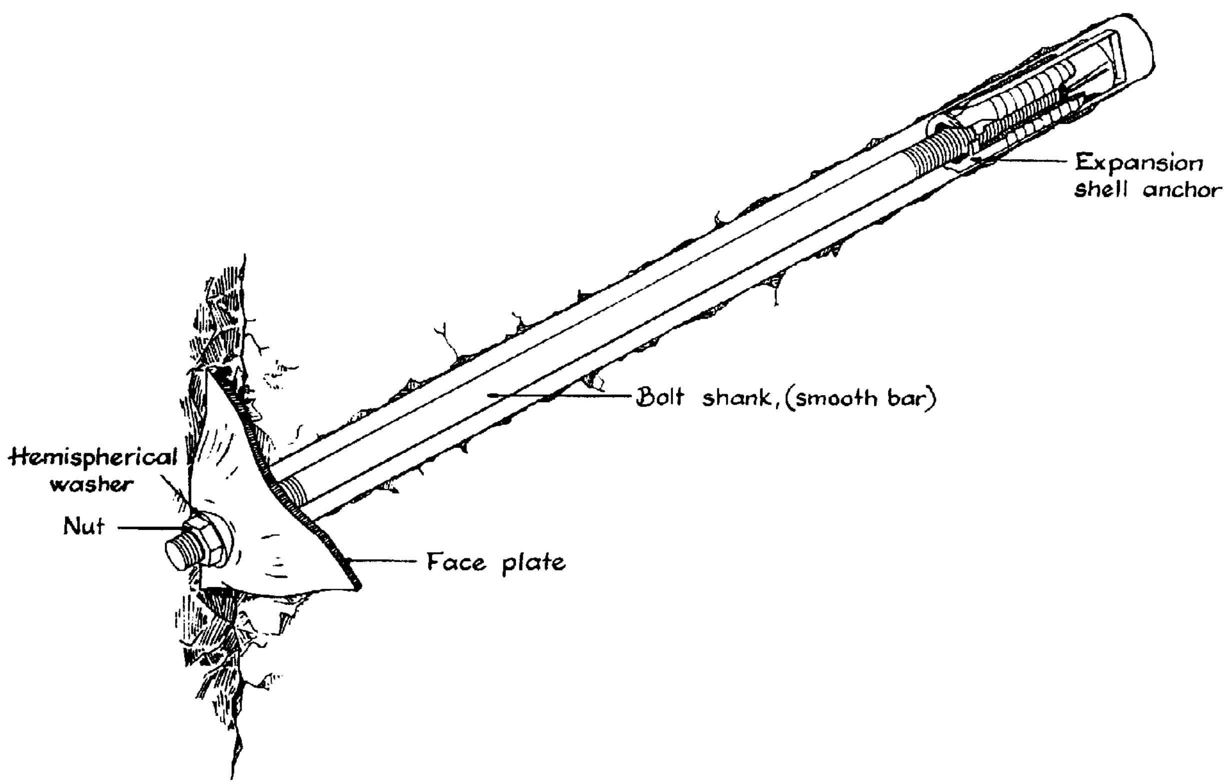



**Figure 5-8. Mechanically anchored rock bolt–expansion shell anchor**

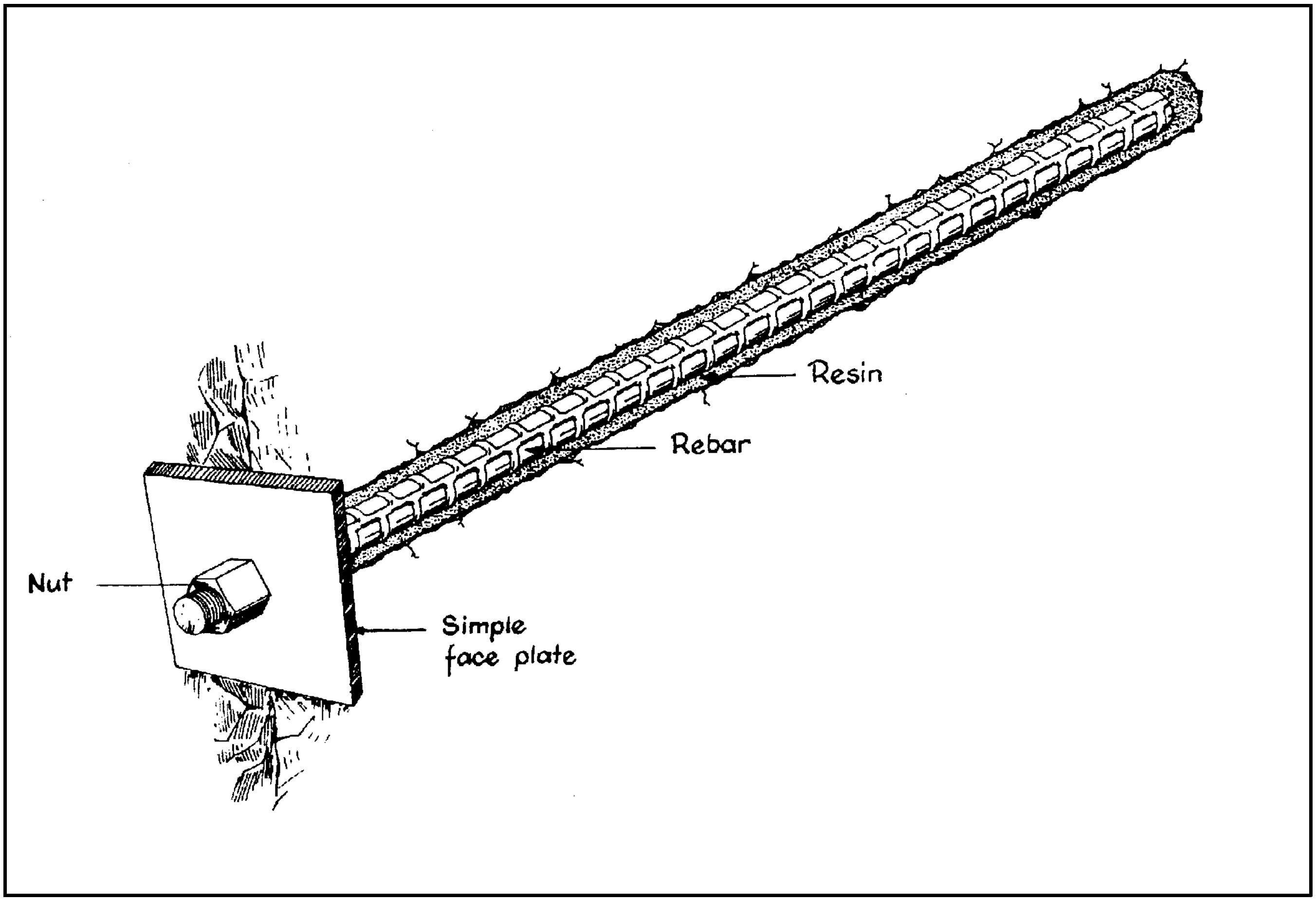



**Figure 5-9. Grouted dowel—rebar**

(1) *Installation.* To install a rock bolt or dowel, a borehole must be drilled into the rock of a specific diameter and length no matter what type of bolt or dowel is being used. This can be accomplished with a jack leg for small installations or a drill jumbo when high productivity is required. Special rock dowel installation gear is often used. In a blasted tunnel, the drill jumbo used for drilling the blast holes is frequently used to drill the rock bolt holes. Except for split sets, the diameter of the rock bolt hole can vary somewhat. It is common to have up to 10 or 20 percent variation in the hole diameter because of movement and vibration of the drill steel during drilling and variations in the rock. For expansion anchors and grouted and Swellex bolts, this is not a serious problem. Split sets are designed for a specific diameter hole, however; if the hole is larger, it will not have the required frictional resistance. Therefore, drilling of the hole for split sets must be closely controlled. After the hole is drilled, it should be cleaned out (usually with an air jet) and the bolt or dowel installed promptly. For mechanically anchored rock bolts, the bolt is preassembled, slid into the hole, and tightened with a torque wrench. The final tension in the bolt should be created by a direct-pull jack, not by a torque wrench. For resin-grouted rock dowels, the grout is placed in the hole using premade two-component cartridges; the bar is installed using a drill that turns the bar, breaks open the cartridges, and mixes the two components of the resin. The time and method of mixing recommended by the manufacturer should be used. Cement-grouted dowels can be installed the same way except that the grout is pumped into the hole through a tube in the center of the bar.

(2) *Tensioning.* Grouted bolts can be left untensioned after installation or can be tensioned using a torque wrench or a hydraulic jack (Figure 5-13) after the grout has reached adequate strength. Fast-set resin grout can be used to hasten the process for resin-grouted bolts. Cement grout takes longer to cure even if an accelerator is used. Rock bolts in tunnels are usually left untensioned after installation and become tensioned as the rock mass adjusts to the changes in stress brought on by the process of excavation.

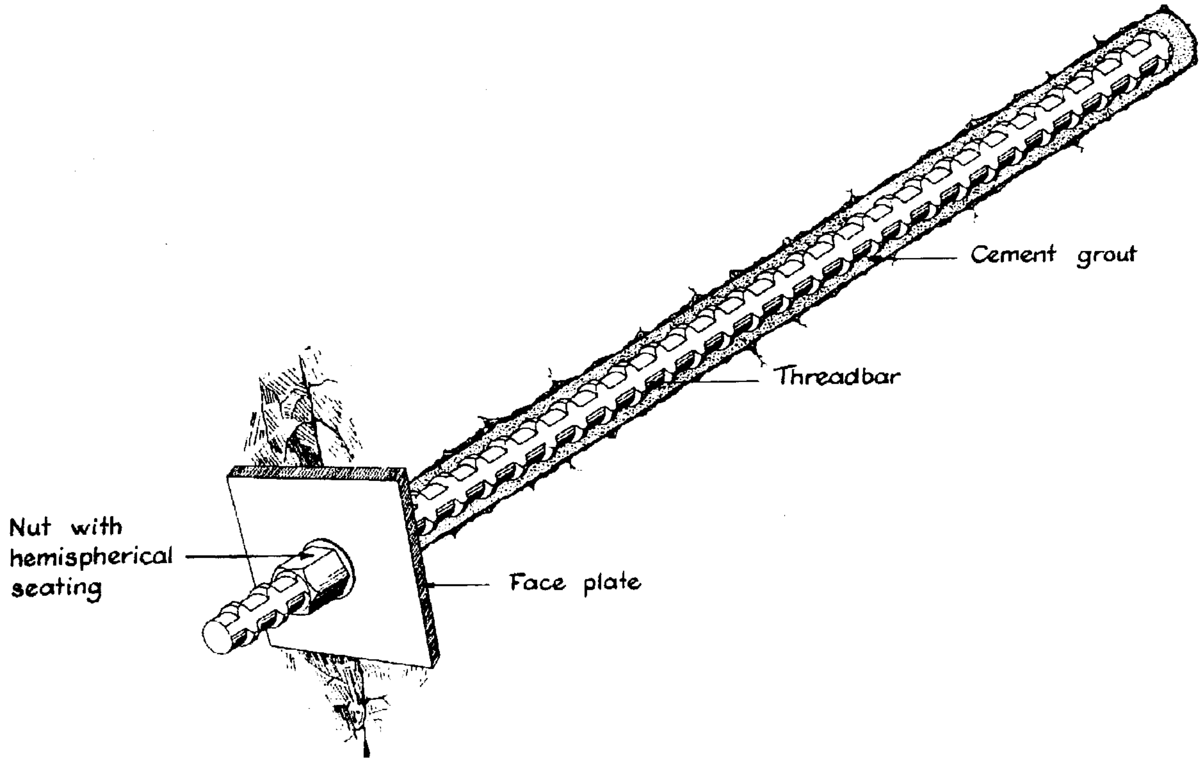



**Figure 5-10. Grouted dowel—Dywidage ® Steel**

Split sets and Swellex bolts work this way since they cannot be pretensioned. There are cases when pretensioning the bolt is necessary, such as to increase the normal force across a joint along which a wedge or block can slip.

(3) *Hardware*. Rock bolts usually have end plates (Figure 5-14) held in place with nuts and washers on the ends of bars or by enlargement of the head of split sets and Swellex bolts. End plates provide the reaction against the rock for tensioned bolts. End plates also are used to hold in place steel mesh and mine straps. They can also be embedded in shotcrete to provide an integral system of rock reinforcement and surface protection (Figure 5-15). End plates are generally square, round, or triangular shaped (Figure 5-16). Steel mesh, mine straps, and shotcrete are used to hold small pieces of rock in place between the rock bolts.

(4) *Testing*. Testing rock bolts is an important part of the construction process. If the rock bolts are not adequately installed, they will not perform the intended function. Possible reasons for faulty installations include the following:

- Incorrect selection of the rock bolt system.
- Incorrect placement of borehole.
- Incorrect length of borehole.
- Incorrect diameter of borehole.
- Inadequate cleaning of borehole.
- Inadequate placement of grout.
- Inadequate bond length of grout.
- Corrosion or foreign material on steel.
- Misalignment of rock bolt nut and bearing plate assembly.

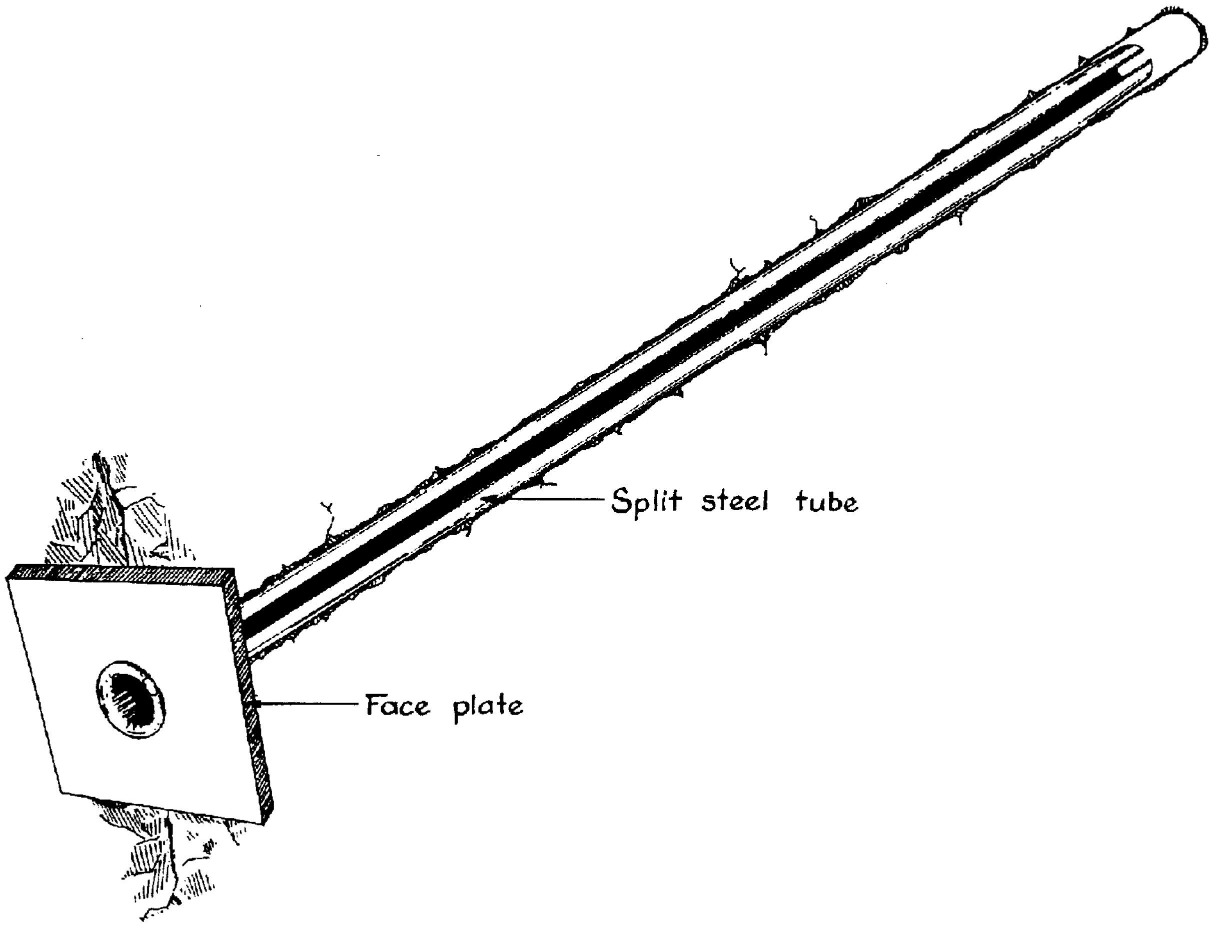



**Figure 5-11. Friction dowel—Split Set ®**

- Out-of-date grouting agents.
- Inappropriate grout mixture.
- Damage to breather tube.
- Inadequate borehole sealing.
- Inadequate lubrication of end hardware.
- Incorrect anchor installation procedure.
- Inadequate test program.
- No monitoring of rock bolt system performance.

Many of these problems can be avoided by adherence to manufacturer installation recommendations, and manufacturer representatives may be required to be onsite at the beginning of rock bolting operations to ensure conformance and trouble-shoot problems. The most common method of testing rock bolts or dowels is the pull-out test. A hydraulic jack is attached to the end of the rock bolt and is used to load the rock bolt to a predetermined tensile load and displacement. Rock bolts may be tested to failure or to a lesser value so that they can be left in place to perform their intended function. If the test load or displacement is exceeded, that rock bolt or dowel has failed and others in the area are tested to see if the failure is an isolated problem or indicative of a systematic problem related to all of the bolts or dowels. Usually, many units are tested at the

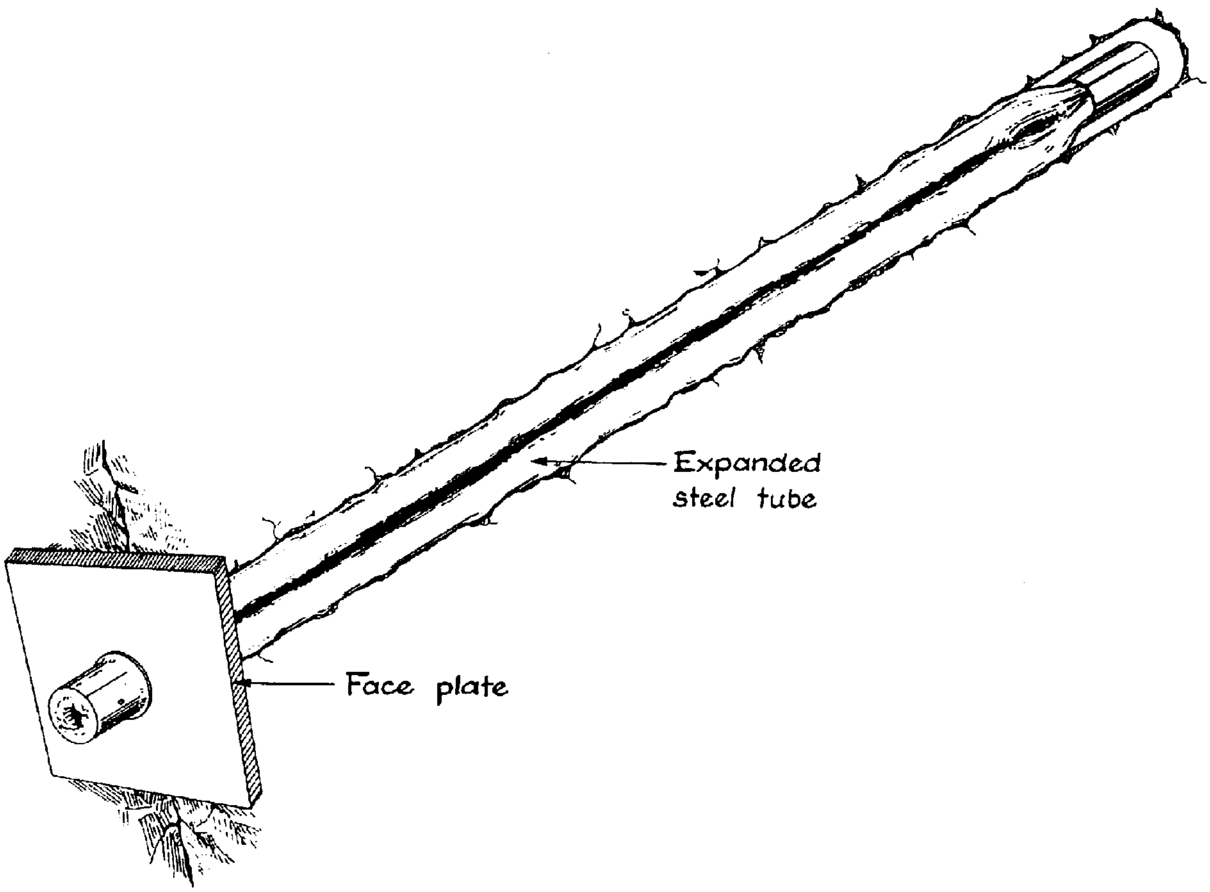



**Figure 5-12. Friction dowel–Swellex ®**

beginning of tunneling, and once installation procedures, methods, and personnel skills are adequately confirmed, then a more moderate testing rate is adopted. If problems occur, changes are made, and a more rigorous testing scheme is reinstated until confidence is restored. Pull-out tests do not test the entire dowel. Only that length of the dowel that is required to resist the pull-out force is tested. For example, a dowel may be only partially grouted and still resist the pull-out force. These uncertainties are generally accepted in tunnel construction, and credence is placed on tunnel performance and pull-out test results. To further test the installation, the dowel can be overcored and exhumed from the rock for direct inspection. However, this requires costly special equipment and is only done under unusual circumstances. Other methods of testing include checking the tightness of a mechanically anchored rock bolt with a torque wrench, installing load cells on the end of tensioned rock bolts, and nondestructive testing by transmitting stress waves down through the bolt from the outer end and monitoring the stress wave return. The less stress wave reflection that is observed, the better the installation is. Swellex bolts can be tested using nondestructive techniques by reattaching the installation pump to the end of the bolt and testing to see that the tube still holds the same amount of pressure as when it was installed.

*c. Shotcrete application*. Shotcrete today plays a vital role in most tunnel and shaft construction in rock because of its versatility, adaptability, and economy. Desirable characteristics of shotcrete include its ability to be applied immediately to freshly excavated rock surfaces and to complex shapes such as shaft and tunnel intersections, enlargements, crossovers, and bifurcations and the ability to have the applied thickness and mix formulation varied to suit variations in ground behavior. A brittle

**Table 5-5**
**Typical Technical Data on Various Rock Bolt Systems**

| Item | Mechanically Anchored | Resin-Grouted Bolts (Rebar) | Cement-Grouted Bolts (Dywidag) | Friction Anchored (Split Set) | Friction Anchored (Swellex) |
|---|---|---|---|---|---|
| Steel quality, MPa | 700 | 570 | 1,080 | Special | Special |
| Steel diameter, mm | 16 | 20 | 20 | 39 | 26 |
| Yield load, steel, kN | 140 | 120 | 283 | 90 | 130 |
| Ultimate load, steel kN | 180 | 180 | 339 | 110 | 130 |
| Ultimate axial strain, steel, % | 14 | 15 | 9.5 | 16 | 10 |
| Weight of bolt steel kg/m | 2 | 2.6 | 2.6 | 1.8 | 2 |
| Bolt lengths, m | Any | Any | Any | 0.9-3 | Any |
| Usual borehole diameter, mm | 35-38 | 30-40 | 32-38 | 35-38 | 32-38 |
| Advantages | Inexpensive. Immediate support. Can be permanent. High bolt loads. | Rapid support. Can be tensioned. High corrosion resistance. Can be used in most rocks. | Competent and durable. High corrosion resistance. Can be used in most rocks. Inexpensive. | Rapid and simple installation. Immediate support. No special equipment. | Rapid and simple installation. Immediate support. Good for variety of conditions. |
| Disadvantages | Use only in hard rock. Difficult to install reliably. Must check for proper tensioning. Can loosen due to blasting. | Messy. Grout has limited shelf life. Sensitive to tunnel environment. | Takes longer to install than resin bolts. Can attain high bolt loads. | Expensive. Borehole diameter crucial. Only short lengths. Not resistant to corrosion. | Expensive. Not resistant to corrosion. Special pump required. |

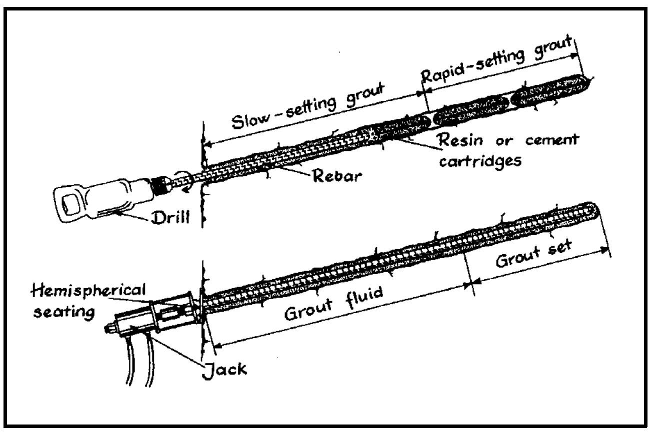



**Figure 5-13. Tension resin dowel installation**

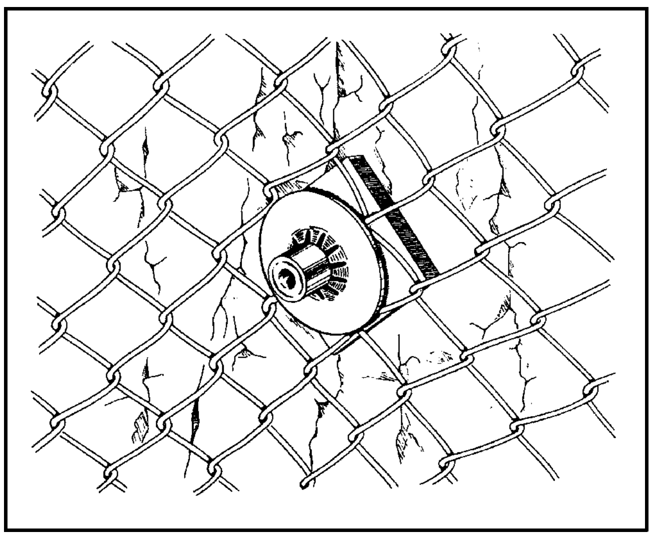


**Figure 5-14. Mesh washer end hardware**

material by nature, shotcrete used for ground support often requires reinforcement to give it strain capacity in tension (i.e., ductility) and to give it toughness. Chain link mesh or welded wire fabric has long served as the method to reinforce shotcrete, but has now been largely supplanted by steel fibers mixed with the cement and the aggregate. Steel fiber reinforced shotcrete (SFRS) was first used in tunnels in North America by the USACE in 1972 in an adit at Ririe Dam (Idaho) (Morgan 1991). In addition to improving toughness and flexural strength, steel fibers improve the fatigue and impact resistance of the shotcrete

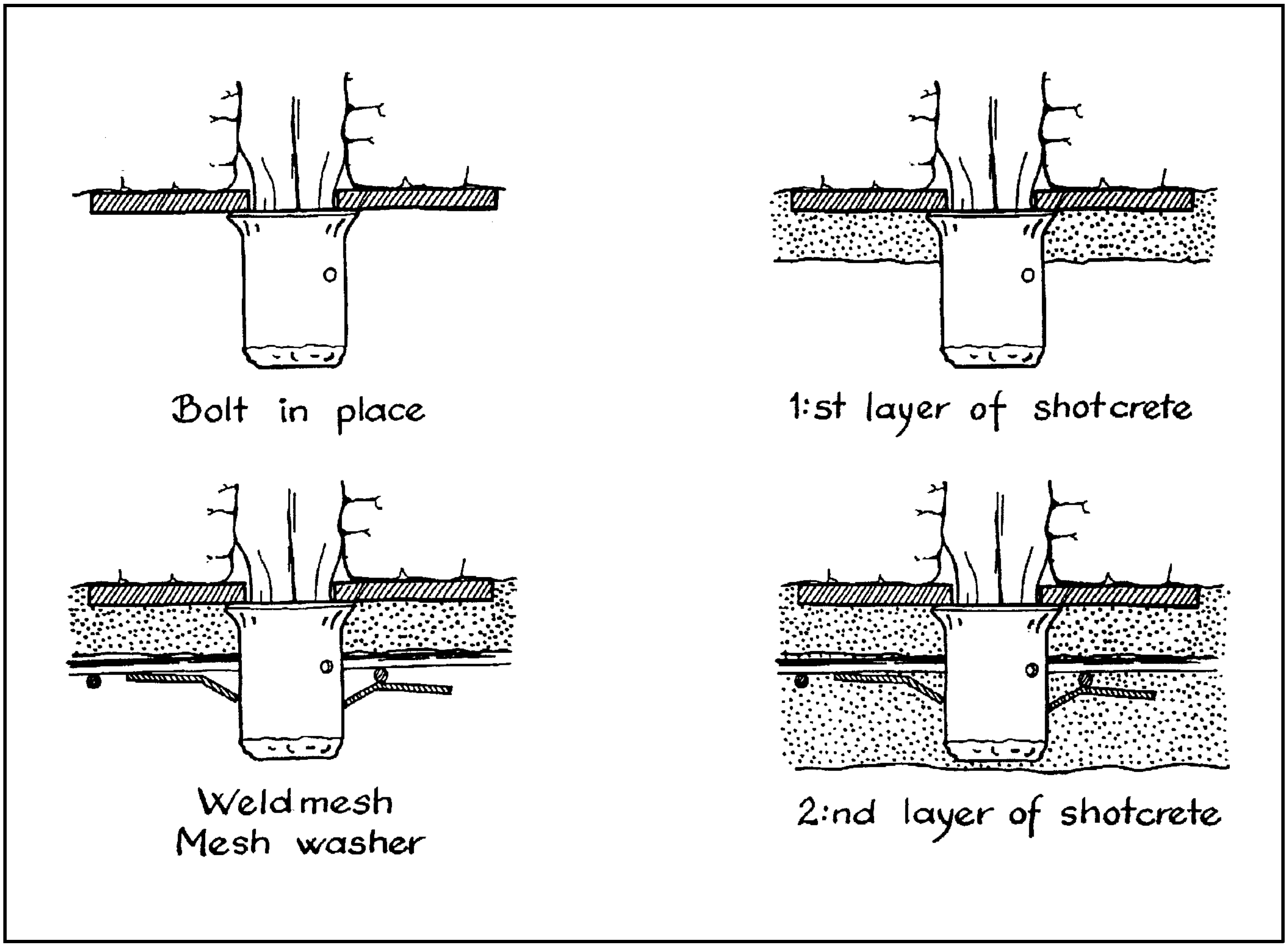



**Figure 5-15. Dowels with end hardware embedded in shotcrete**

layer. Other relatively recent improvements to shotcrete applications include admixtures for a variety of purposes, notable among which is the use of microsilica, which greatly reduces rebound and increases density, strength, and water tightness. EM 1110-2-2005 provides guidance in the design and application of shotcrete.

(1) *Range of applications*. For most tunnels and shafts, shotcrete is used as an initial ground support component. It is sprayed on freshly exposed rock in layers 2 to 4 in. thick where it sets in a matter of minutes or hours, depending on the amount of accelerator applied, and helps support the rock. In blasted rock with irregular surfaces, shotcrete accumulates to greater thicknesses in the overbreaks. This helps prevent block motion and fallout due to shear, by adhering to the irregular surface. On more uniform surfaces, the shotcrete supports blocks by a combination of shear, adhesion, and moment resistance and supports uniform and nonuniform radial loads by shell action and adhesion. By helping prevent the initiation of rock falls, shotcrete also prevents loosening of the rock mass and the potential for raveling failure. Shotcrete also protects surfaces of rock types that are sensitive to changes of moisture content, such as swelling or slaking rock. The application of shotcrete is an essential ingredient in the construction method of sequential excavation and support, where it is used in combination with rock bolts or dowels and, sometimes, steel ribs or lattice girders in poor ground. For TBM tunnels, initial ground support usually consists of dowels, mesh, mine straps, channels, or steel ribs; shotcrete can be applied some distance behind the advancing face. Only in a few instances have TBMs been built with the possibility to apply shotcrete a short distance behind the face.

(2) *Reinforced shotcrete*. In poor or squeezing ground, additional ductility of the shotcrete is desirable.

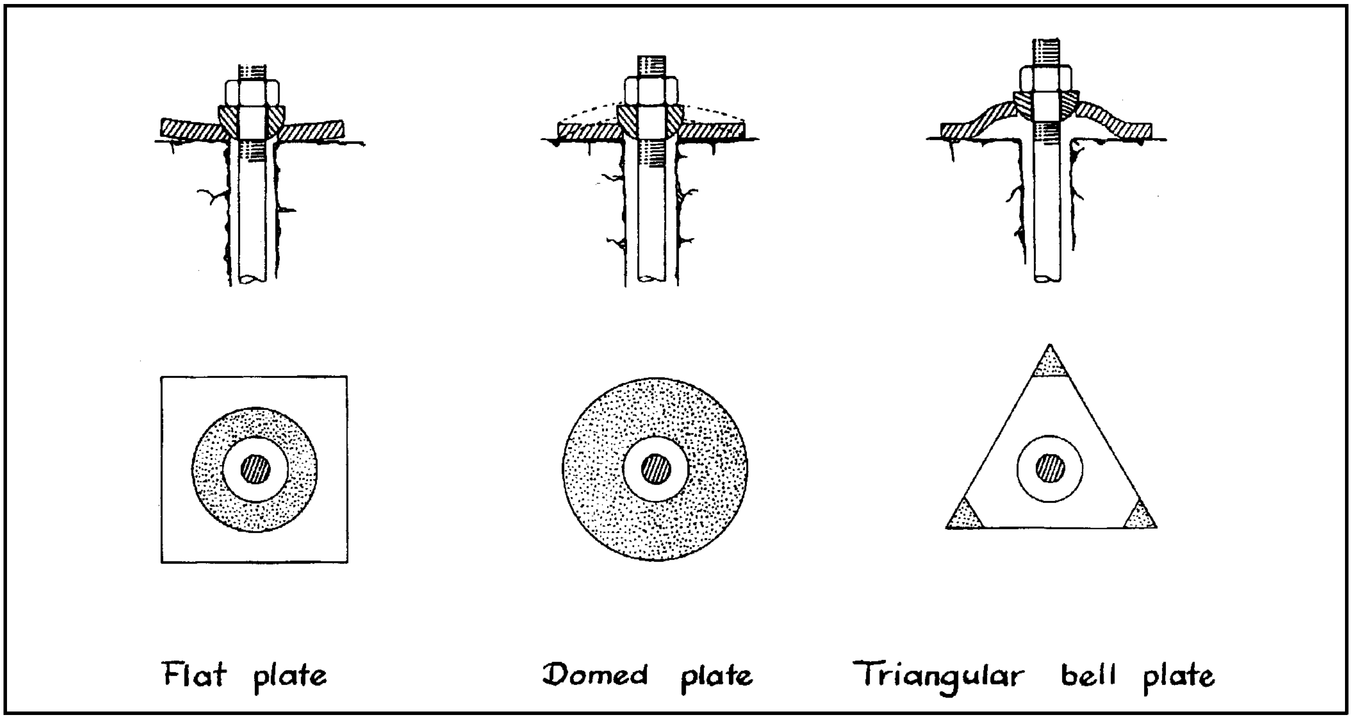



**Figure 5-16. Different types of end hardware**

Until recently this ductility was generally achieved by welded wire fabric usually applied between the first and the second coat of shotcrete. While wire fabric does add to the ductility of the shotcrete, it has several disadvantages. It is laborious and costly to place; it is difficult to obtain good shotcrete quality around and behind wires; and it often results in greater required shotcrete volumes, because the fabric cannot be draped close to the rock surface on irregularly shot surfaces. Modern reinforced shotcrete is almost always steel fiber-reinforced shotcrete. The steel fibers are generally 25- to 38-mm-long deformed steel strips or pins, with an aspect ratio, length to width or thickness, between 50 and 70. These steel fibers are added to the shotcrete mix at a rate of 50-80 kg/m$^3$ (85-135 lb/yd$^3$) without any other change to the mix. The steel fibers increase the flexural and tensile strength but more importantly greatly enhance the postfailure ductility of the shotcrete. Steel fibers are made and tested according to ASTM A 820 and steel fiber shotcrete according to ASTM C 1116.

*d. Steel ribs and lattice girders.* Installing steel and wooden supports in a tunnel is one of the oldest methods in use. Many years ago, wooden supports were used exclusively for tunnel support. In later years, steel ribs (Figure 5-17) took the place of wood, and, most recently, steel lattice girders (Figure 5-18) are being used in conjunction with shotcrete. Figure 5-19 shows an application of shotcrete, lattice girders, and dowels for a rapid transit tunnel through a fault zone. It is usually faster and more economical to reinforce the rock with rock bolts, steel mesh or straps, and shotcrete so the rock will support itself. However, if the anticipated rock loads are too great, such as in faulted or weathered ground, steel supports may be required. Steel ribs and lattice girders usually are installed in the tunnel in sections within one rib spacing of the tunnel face. The ribs are generally assembled from the bottom up making certain that the rib has adequate footing and lateral rigidity. Lateral spacer rods (collar braces) are usually placed between ribs to assist in the installation and provide continuity between ribs. During and after the rib is erected, it is blocked into place with grout-inflated sacks as lagging, or shotcrete. In modern tunnel practice, the use of wood blocking is discouraged because it is deformable and can deteriorate with time. The rib functions as an arch, and it must be confined properly around the perimeter. The manufacturer of steel ribs provides recommendations concerning the spacing of blocking points that should be followed closely (see Proctor and White 1946). When shotcrete is used as lagging, it is important to make sure that no voids or laminations are occurring as the shotcrete spray hits the steel elements. Steel ribs should be fully embedded in the shotcrete. The lattice girders are filled in by shotcrete in addition to being embedded in shotcrete. Steel ribs and lattice girders are often not the

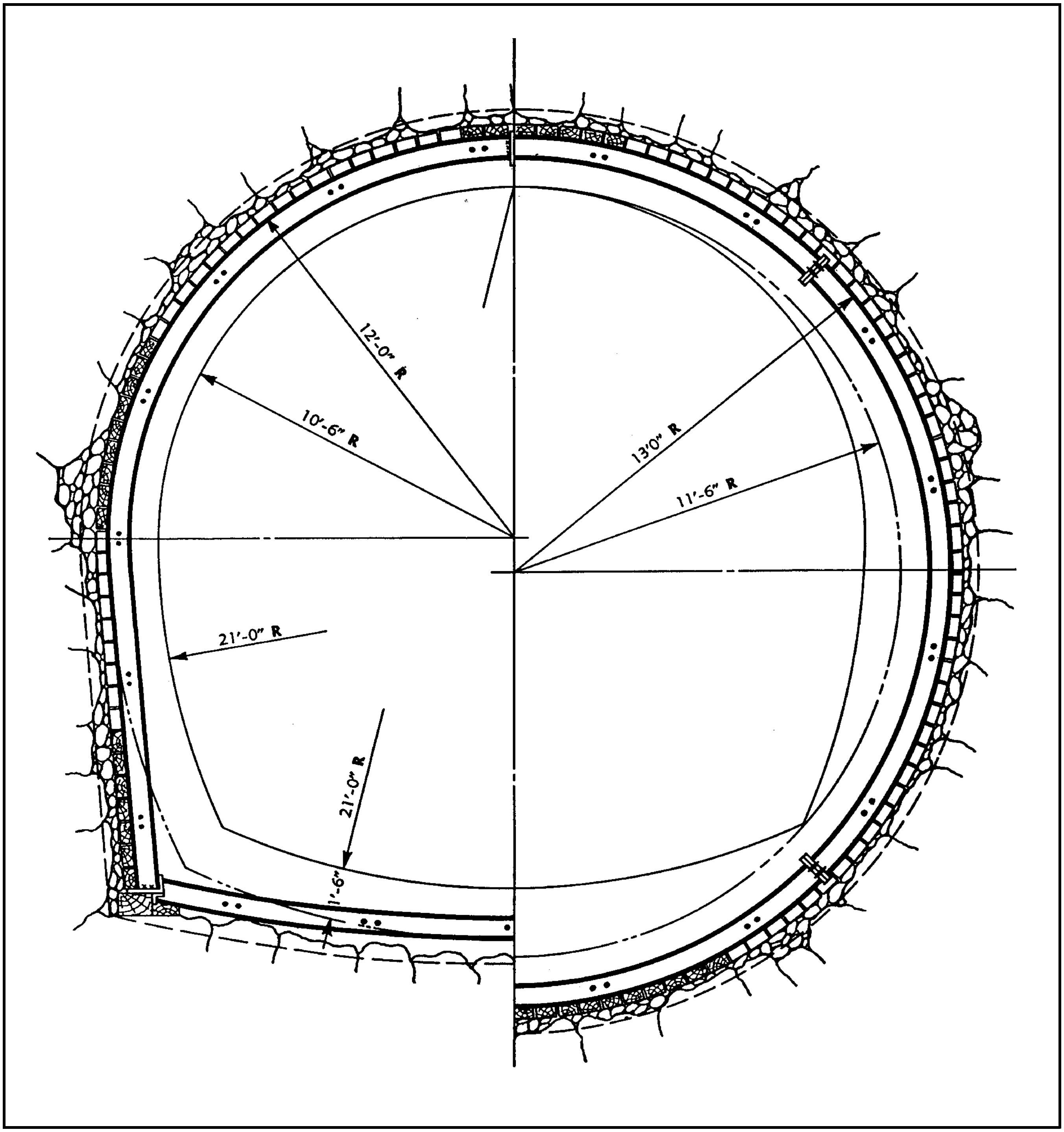



**Figure 5-17. Steel rib examples, conversion of a horseshoe-shaped flow tunnel to a circular shape in squeezing ground**

sole method of tunnel support but are only provided in the event that bad tunneling conditions are encountered. In this case, it is necessary to have all the required pieces at the site and have adequately trained personnel ready when they are required in order to reduce delays in switching to a different type of tunnel support.

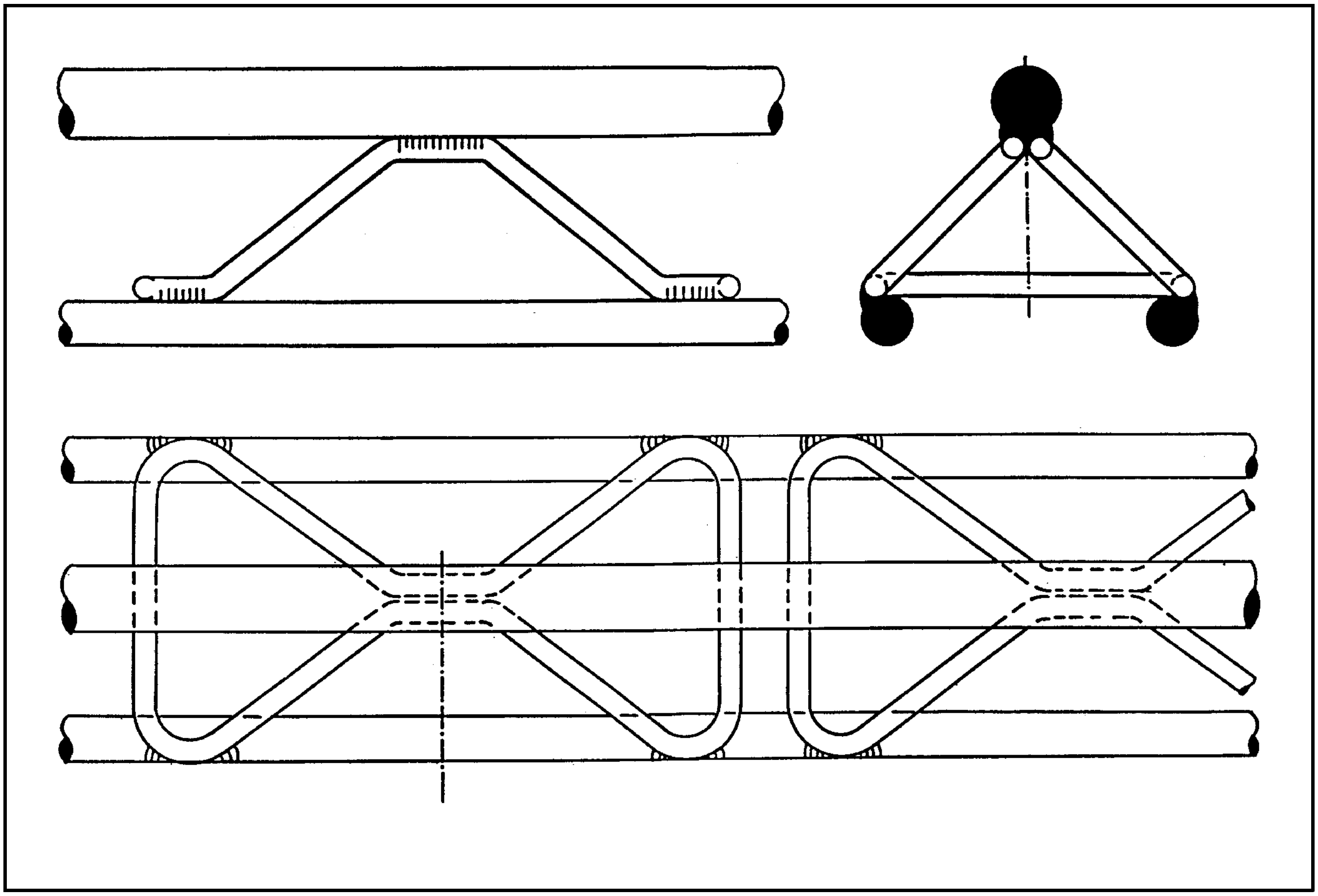


**Figure 5-18. Lattice girders**

*e. Precast concrete segments used with TBM.* Soft ground tunnels in the United States are most often constructed using shields or shielded TBMs with precast concrete segments. Below the groundwater table, the segments are bolted with gaskets for water tightness. Above the groundwater table, unbolted, expanded segmental linings are often used, followed by a cast-in-place concrete lining (two-pass lining). If necessary, a water- or gasproofing membrane is placed before the cast-in-place concrete is placed. The shield or TBM is usually moved forward using jacks pushing on the erected segmental concrete lining. Hard rock tunnels driven with a TBM may also be driven with some form of segmental lining, either a one-pass or two-pass lining. There are several reasons for this choice.

(1) For the completion of a long tunnel, the schedule may not permit the length of time required to cast a lining in place. The option of casting lining concrete while advancing the TBM is feasible, at least for a large-diameter tunnel, but often not practical. Interference between concrete transportation and placement and tunnel excavation and mucking is likely to slow tunnel driving. Transporting fresh concrete for a long distance can also be difficult. In this instance, placing a one-pass segmental lining is a practical solution, provided that lining erection does not significantly slow the advance of the TBM.

*f. Bolted or unbolted segments.* A gasketed and bolted segmental lining must be fabricated with great precision, and bolting extends the time required for erection. Hence, such a lining is usually expensive to manufacture and to erect. For most water tunnels, and for many other tunnels, a fully gasketed and bolted, watertight lining is not required, and an unbolted segmental lining is adequate.

*g. Segment details.* Once a segmental lining has been determined to be feasible or desirable, the designer has a number of choices to make. In the end, the contractor may propose a different lining system of equal quality that better fits his/her proposed methods of installation. A

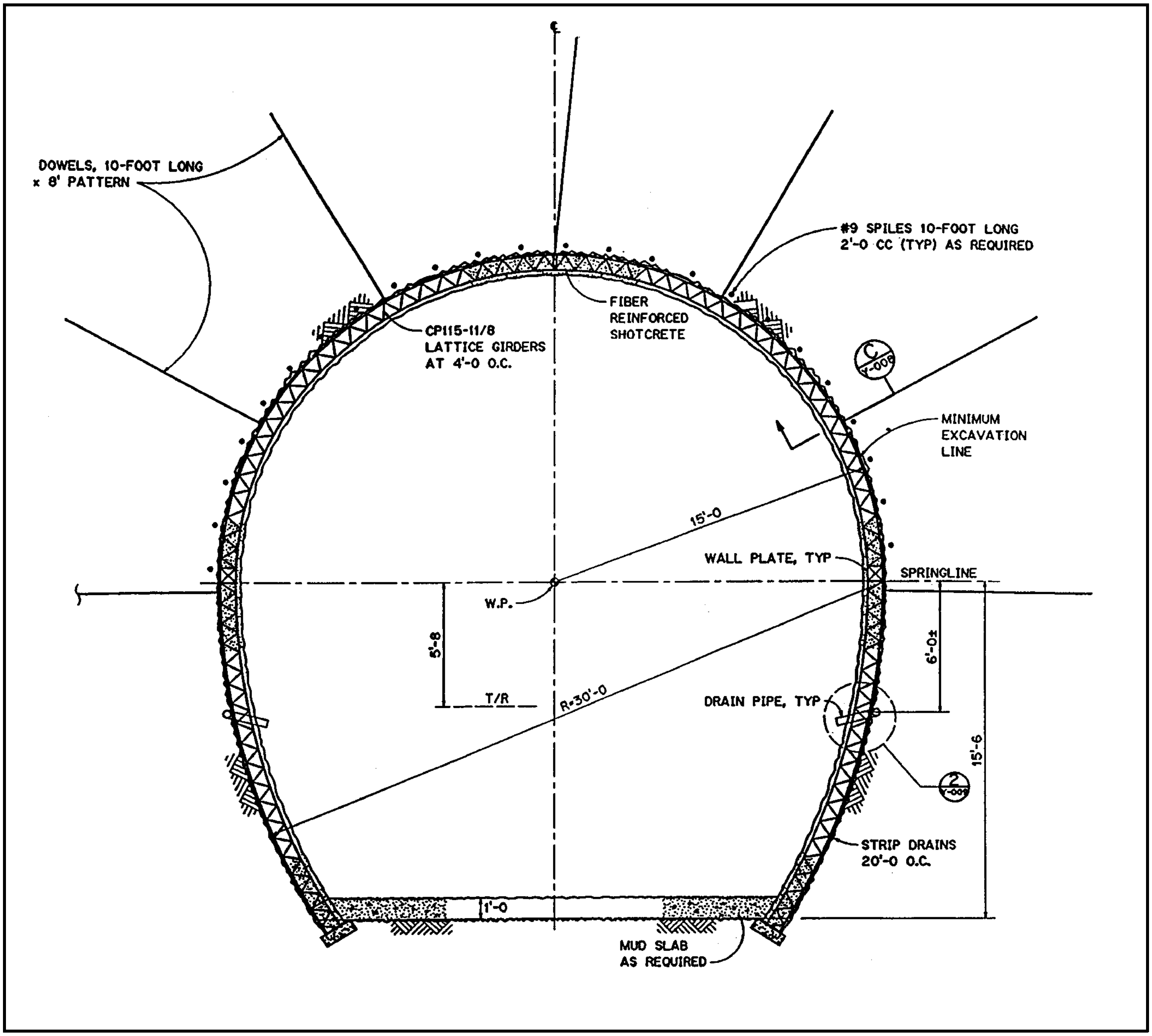



**Figure 5-19. Lattice girders used as final support with steel-reinforced shotcrete, dowels, and spiles**

selection of lining and joint details are shown on Figures 5-20 to 5-22. Details are selected to meet functional requirements, and for practicality and economy of construction. For the most part, details can be mixed liberally to match given requirements and personal preferences.

*h. Matching construction methods and equipment.* When a tunnel lining system has been selected, construction methods and equipment must be designed to match the specific needs of this system. With a full shield tail, the invert segment is placed on the shield surface at the bottom. When the shield passes, the invert segment falls to the bottom, unless it is bolted to the previous segment. The erector equipment must match the pick-up holes in the segments, be able to rotate the segment into its proper place, and must have all of the motions (radial, tangential, axial, tilt, etc.) to place the segment with the tolerances required. Relatively high speed motion is required to bring a segment to its approximate location, but inching speed is often required for precise positioning. Unless each segment is stable as placed, holding devices are required to prevent them from falling out until the last segment is in place. Such holding devices are not required for a bolted and for most dowelled linings.

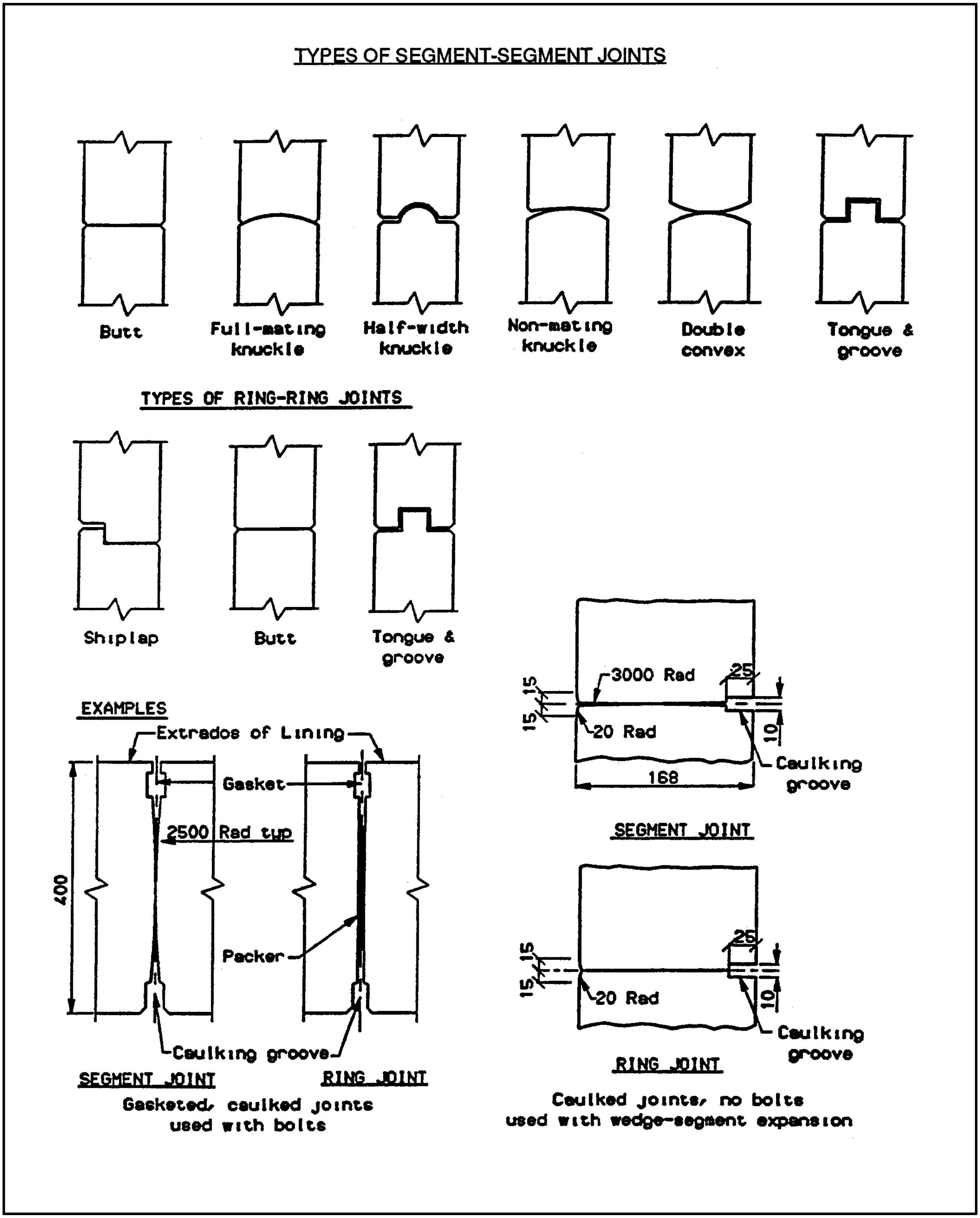



Figure 5-20. Types of joints in segmental concrete lining

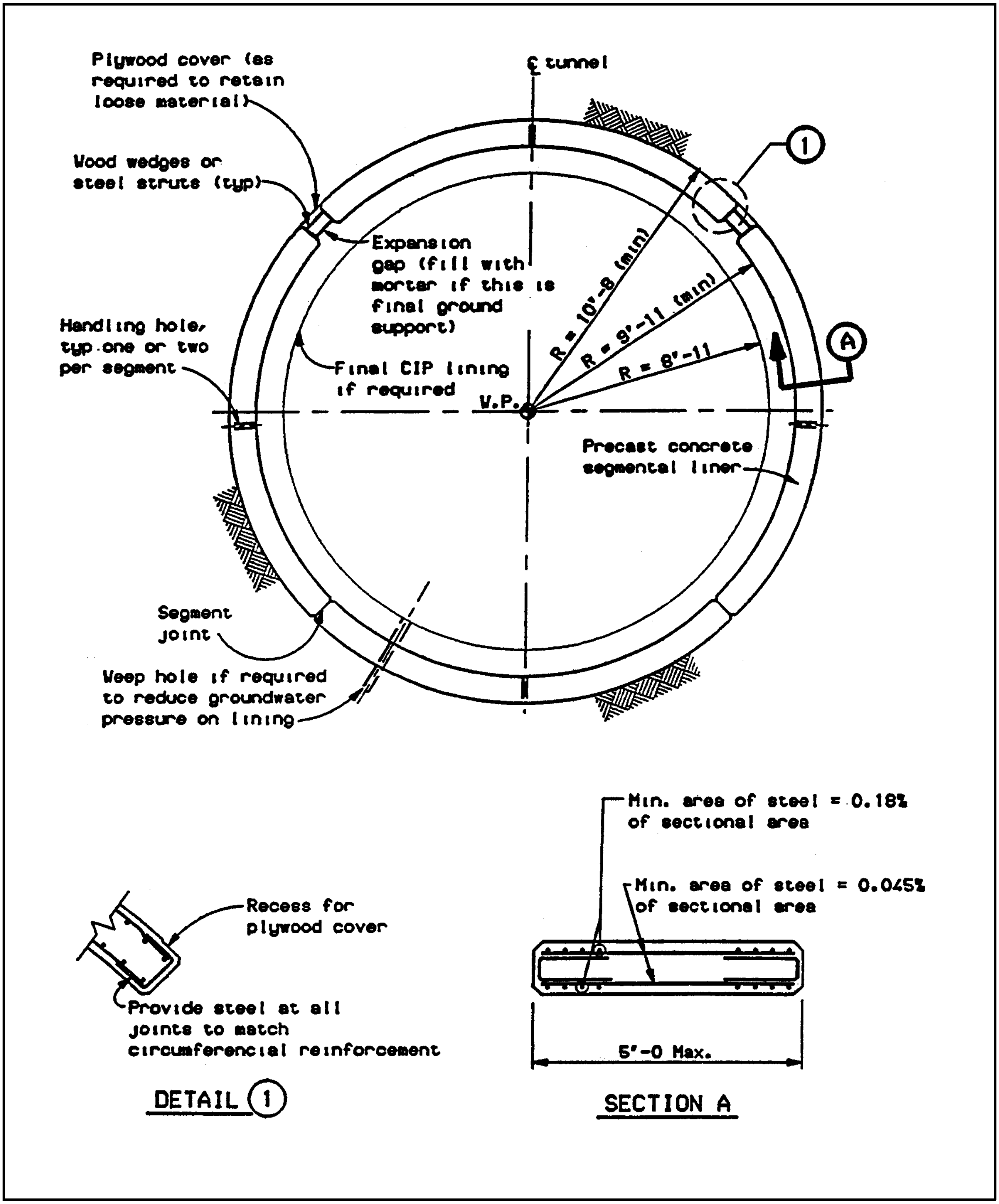



Figure 5-21. Simple expanded precast concrete lining used as initial ground support or as final ground support

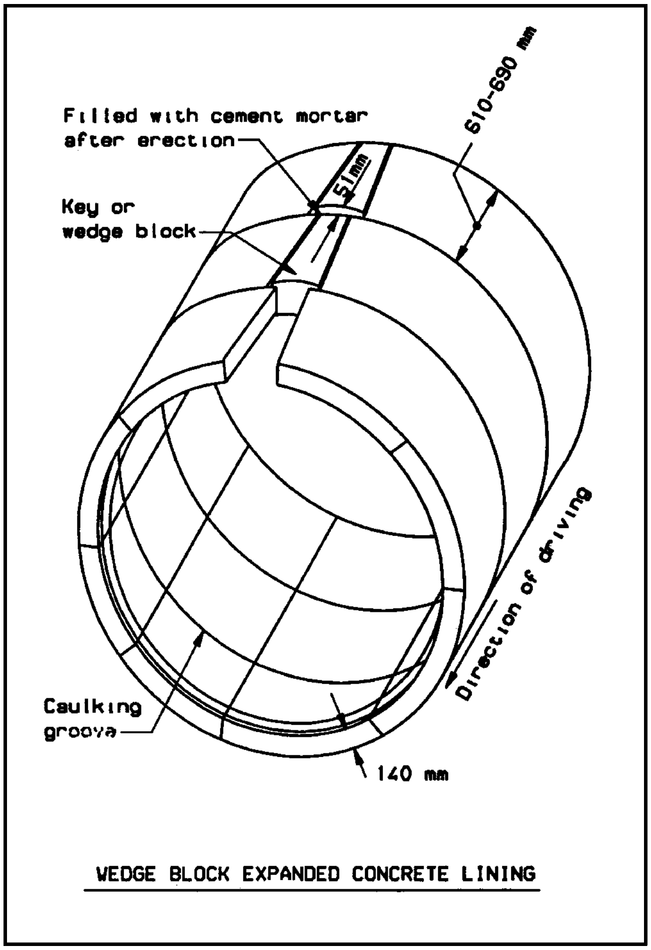



**Figure 5-22. Wedge block expanded concrete lining**

*i. Functional criteria for one-pass segmental linings.*

(1) Selection of a segmental lining system is based on considerations of cost and constructibility, and many details depend on the construction procedure. Functional criteria, however, must also be met.

(2) Water flow and velocity criteria often require a smooth lining to achieve a reasonably low Mannings number. This may require limitations on the offset permitted between adjacent segment rings. With an expanded lining, it is often not possible to obtain full expansion of all rings, and offsets between rings can be several centimeters. If this is not acceptable, an unexpanded dowelled or bolted ring may be required.

(3) In the event that some segments are, in fact, erected with unacceptable offsets, the hydraulic effect can be minimized by grinding down the protrusions or filling the shadows.

(4) A watertight lining is difficult to obtain using segments without gaskets. In some lining systems, sealing strips or caulking are employed to retain grout filling, but cannot sustain high groundwater pressures. In wet ground, it may be necessary to perform formation grouting to reduce water flows. Alternatively, fully gasketed and bolted linings may be used through the wet zones. This choice depends on the acceptability of water into or out from the tunnel during operations and the differential water pressure between the formation and the tunnel. The choice also depends on the practicality and economy of grouting during construction.

(5) The lining segments must be designed to withstand transport and construction loads. During storage and transport, segments are usually stacked with strips of timber as separation. Invert segments must withstand uneven loads from muck trains and other loads. The design of invert segments must consider that the segments may not be perfectly bedded. Lining rings used as reaction for shield propulsion must be able to withstand the distributed loads from the jacks, including eccentricities resulting from mismatching adjacent rings.

(6) Joint details must be reinforced to resist chipping and spalling due to erection impact and the effect of uneven jacking on imprecisely placed segments. Tongue-and-groove joints are particularly susceptible to spalling, and the edges of the groove may require reinforcement.

(7) Permanence of the finished structure requires consideration of long-term corrosion and abrasion effects. For a one-pass segmental lining, a high-strength concrete with a high pozzolan replacement is usually desirable for strength, density, tightness, and durability. Precast concrete of 41.4 MPa (6,000-psi) (28-day cylinder) strength or more is routinely used for this purpose. Reinforcement should be as simple as possible, preferably using prefabricated wire mesh.

(8) Once construction and long-term performance requirements have been met, postulated or actual exterior ground or water loads are usually of minor consequence. In rare instances, squeezing ground conditions at great depth may require a thicker lining or higher concrete strength. Water pressures may be reduced by deliberately permitting seepage into the tunnel, and moments in the lining are reduced by using unbolted joints.

## 5-5. Sequential Excavation and Support

Recognizing the inherent variability of geologic conditions, several construction methods have been developed so that methods of excavation and support can be varied to suit encountered conditions. The most famous of these methods is the New Austrian Tunneling Method (NATM), developed and commonly used in Central Europe. Much older, and applied throughout the world, is the observational method. Both of these methods are discussed in the following sections.

*a. NATM.*

(1) The so-called NATM is employed for large, noncircular tunnels in poor ground where ground support must be applied rapidly. NATM usually involves the following components:

- Heading-and-bench or multidrift excavation (no shield or TBM).

  Excavation by blasting or, more commonly, by roadheader or other mechanical means.

- Initial ground support usually consisting of a combination of shotcrete, dowels, steel sets, or (now more commonly) lattice girders, installed quickly after exposure by excavation.

- Forepoling or spiling where the ground requires it.

- Stabilizing the face temporarily, using shotcrete and possibly glass-fiber dowels.

- Ground improvement (grouting, freezing, dewatering).

  Extensive use of monitoring to ascertain the stability and rate of convergence of the opening.

(2) The final lining usually consists of reinforced, cast-in-place concrete, often with a waterproofing membrane between the cast-in-place concrete and the initial ground support.

(3) It would appear that the NATM employs virtually all of the means and methods available for tunneling through poor ground. What distinguishes the method is the extensive use of instrumentation and monitoring as an essential part of the construction method. Traditionally, monitoring involves the use of the following devices (see Chapter 9 for additional information about instrumentation and monitoring):

- Convergence measurements, wall to wall and wall to crown.

- Surveying techniques, floor heave, crown sag.

- Multiposition borehole extensometers.

- Strain gages or load cells in the shotcrete, at the rock-shotcrete interface, or on dowels or steel sets, or lattice girders.

(4) The instrumentation is used to assess the stability and state of deformation of the rock mass and the initial ground support and the buildup of loads in or on support components. In the event that displacements maintain their rate or accelerate, that loads build to greater values than support components can sustain, or if instability is visually observed (cracks, distortion), then additional initial ground support is applied. Final lining is placed only after ground movements have virtually stopped.

(5) Initial ground support intensity (number of dowels, thickness of shotcrete, and spacing of steel sets or lattice girders) is applied according to conditions observed and supplemented as determined based on monitoring data. The overall cross section can also be varied according to conditions, changing from straight to curved side walls. The invert can be overexcavated to install a straight or downward curved strut when large lateral forces occur. In addition, sequences of excavation can be changed, for example from heading-and-bench excavation to multiple drifting.

(6) The NATM has been used successfully for the construction of large tunnel cross sections in very poor ground. On a number of occasions, the method has been used even for soft-ground tunnel construction, sometimes supplemented with compressed air in the tunnel for groundwater control and to improve the stand-up time of the ground. Using the NATM in poor rock requires careful execution by contractor personnel well experienced in this type of work. In spite of careful execution, the NATM is not immune to failure. A number of failures, mostly at or near the tunnel face, have been recorded. These have occurred mostly under shallow cover with unexpected geologic or groundwater conditions or due to faulty application insufficient shotcrete strength or thickness, belated placement of ground support, or advancing the excavation before the shotcrete has achieved adequate strength.

(7) It is common to model the complete sequence of excavation and construction using a finite element or finite differences model so as to ascertain that adequate safety factors are obtained for stresses in the final lining. Elastic or inelastic representations of the rock mass properties are used, and tension cracks in unreinforced concrete or shotcrete that propagate to the middle of the cross section are acceptable.

(8) The NATM method of construction requires a special contract format to permit payment for work actually required and carried out and a special working relationship between the contractor and the owner's representative onsite to agree on the ground support required and paid for. Writing detailed and accurate specifications for this type of work is difficult.

(9) While commonly used in Central Europe, the NATM has not been popular in the United States for a number of reasons:

(a) Ground conditions are, for the most part, better in the United States than in those areas of Europe where NATM is popular. In recent years, there have been few opportunities to employ the NATM in the United States.

(b) Typical contracting practices in the United States make this method difficult to administer.

(c) Emphasis in the United States has been on high-speed, highly mechanized tunneling, using conservative ground support design that is relatively insensitive to geologic variations. NATM is not a high-speed tunneling method.

(d) Most contractors and owners in the United States are not experienced in the use of NATM.

This is not meant to imply that the method should not be considered for use in the United States. Short tunnels or chambers (example: underground subway station) located in poor ground that requires rapid support may well be suited for this method. More often, however, the instrumentation and monitoring component of the NATM is dispensed with or relegated to a minor part of the construction method, perhaps applicable only to limited areas of known difficulty. This type of construction is more properly termed “sequential excavation and support.

*b. The observational method and sequential excavation and support.*

(1) Sequential excavation and support can incorporate some or most of the NATM components, but instrumentation and monitoring are omitted or play a minor role. Instead, a uniform, safe, and rapid excavation and support procedure is adopted for the project for the full length of the tunnel. Or several excavation and support schemes are adopted, each applicable to a portion of the tunnel. The typical application employs a version of the observational method, as follows:

(a) Based on geologic and geotechnical data, the tunnel profile is divided into three to five segments of similar rock quality, where similar ground support can be applied.

(b) Excavation and initial ground support schemes are designed for each of the segments. Excavation options may include full-face advance, heading-and-bench, or multiple drifting. The initial support specification should include designation of maximum time or length of exposure permitted before support is installed.

(c) A method is devised to permit classification of the rock conditions as exposed, in accordance with the excavation and ground support schemes worked out. Sometimes a simplified version of the Q-method of rock mass classification is devised.

(d) Each ground support scheme is priced separately in the bid schedule, using lengths of tunnel to which the schemes are estimated to apply.

(e) During construction the ground is classified as specified, and the contractor is paid in accordance with the unit price bid schedule. The final price may vary from the bid, depending on the actual lengths of different ground classes observed.

(2) The term “sequential excavation and support” is usually employed for excavations that may involve multiple drifting and rapid application of initial support. The observational method works well with this type of construction. However, the observational method also works well with tunneling using TBM. Here, the opening is typically circular, and the initial ground support options do not usually include rapid application of shotcrete, which is considered incompatible with most TBMs. The following is an example of the observational method specified for a TBM-driven tunnel.

(3) Based on the NGI Q-classification system, the rock mass for the Boston Effluent Outfall Sewer Tunnel was divided into three classes: Class A for Q > 4; Class B for 4 > Q > 0.4; and Class C for Q < 0.4. Considering that there would be little time and opportunity to permit continuing classification of the rock mass according to the Q-system, a simplified description was adopted for field use:

- Class A typical lower bound description: RQD = 30 percent, two joint sets (one of which associated with bedding planes) plus occasional random joints, joints rough or irregular, planar to undulating, unaltered to slightly altered joint walls, medium water inflow.

- Class B typical lower bound description: RQD = 10 percent, three joint sets, joints slickensided and undulating, or rough and irregular but planar, joint surfaces slightly altered with nonsoftening coatings, large inflow of water.

  Class C applies to rock poorer than Class B.

(4) With a TBM-driven tunnel, shotcrete was considered inappropriate, particularly since the types of rock expected would not suffer slaking or other deterioration upon exposure. Maximum use was made of rock dowels, wire mesh, and straps in the form of curved channels, as shown on Figure 5-23 to 5-25. Class A rock might in most instances require no support for the temporary condition; nonetheless, initial ground support was specified to add safety and to minimize the effort required for continuous classification of the rock mass.

(5) The contract also provided for having a number of steel sets on hand for use in the event that bolts or dowels are ineffective in a particular reach. Estimates were made for bidding purposes as to the total aggregate length of tunnel for which each rock class was expected, without specifying where.

(6) For the same project, a short length of smaller tunnel was required to be driven by blasting methods. Two classes of rock were introduced here, equivalent to Class A and Classes B + C (very little if any Class C rock was expected here). Ground supports for these rock classes in the blasted tunnel are shown in Figure 5-26.

## 5-6. Portal Construction

*a*. Tunnels usually require a minimum of one to two tunnel diameters of cover before tunneling can safely commence. An open excavation is made to start, which when finished will provide the necessary cover to begin tunneling. Rock reinforcement systems are often used to stabilize the rock cut above the tunnel and are usually combined with a prereinforcement system of dowels installed around the tunnel perimeter to facilitate the initial rounds of excavation (Figure 5-27). If a canopy is to be installed outside of the tunnel portal for protection from rock falls, it should be installed soon after the portal excavation has been completed. If multiple stage tunnel excavation is to be used on the project, the contractor may excavate the portal only down to the top heading level and commence tunneling before taking the portal excavation down to the final grade.

*b*. Tunnel excavation from the portal should be done carefully and judiciously. Controlled blasting techniques should be used and short rounds of about 1 m in depth are adequate to start. After the tunnel has been excavated to two or three diameters from the portal face, or as geology dictates, the blasting rounds can be increased progressively to standard length rounds used for normal tunneling.

*c*. When constructing portals, the following special issues should be accounted for:

(1) The rock in the portal is likely to be more weathered and fractured than the rock of the main part of the tunnel.

(2) The portal must be designed with proper regard for slope stability considerations, since the portal excavation will unload the toe of the slope.

(3) The portal will be excavated at the beginning of mining before the crew has developed a good working relationship and experience.

(4) The slope must be adequately designed to adjust to unloading and stress relaxation deformations.

(5) The portal will be a heavily used area, and a conservative design approach should be taken because of the potential negative effects instability would have on the tunneling operations.

*d*. The design of portal reinforcement will depend on geologic conditions. Rock slope stability methods should be used unless the slope is weathered or under a deep layer of overburden soil. In this case, soil slope stability analyses must be performed for the soil materials. Often, both types of materials are present, which will require a combined analysis.

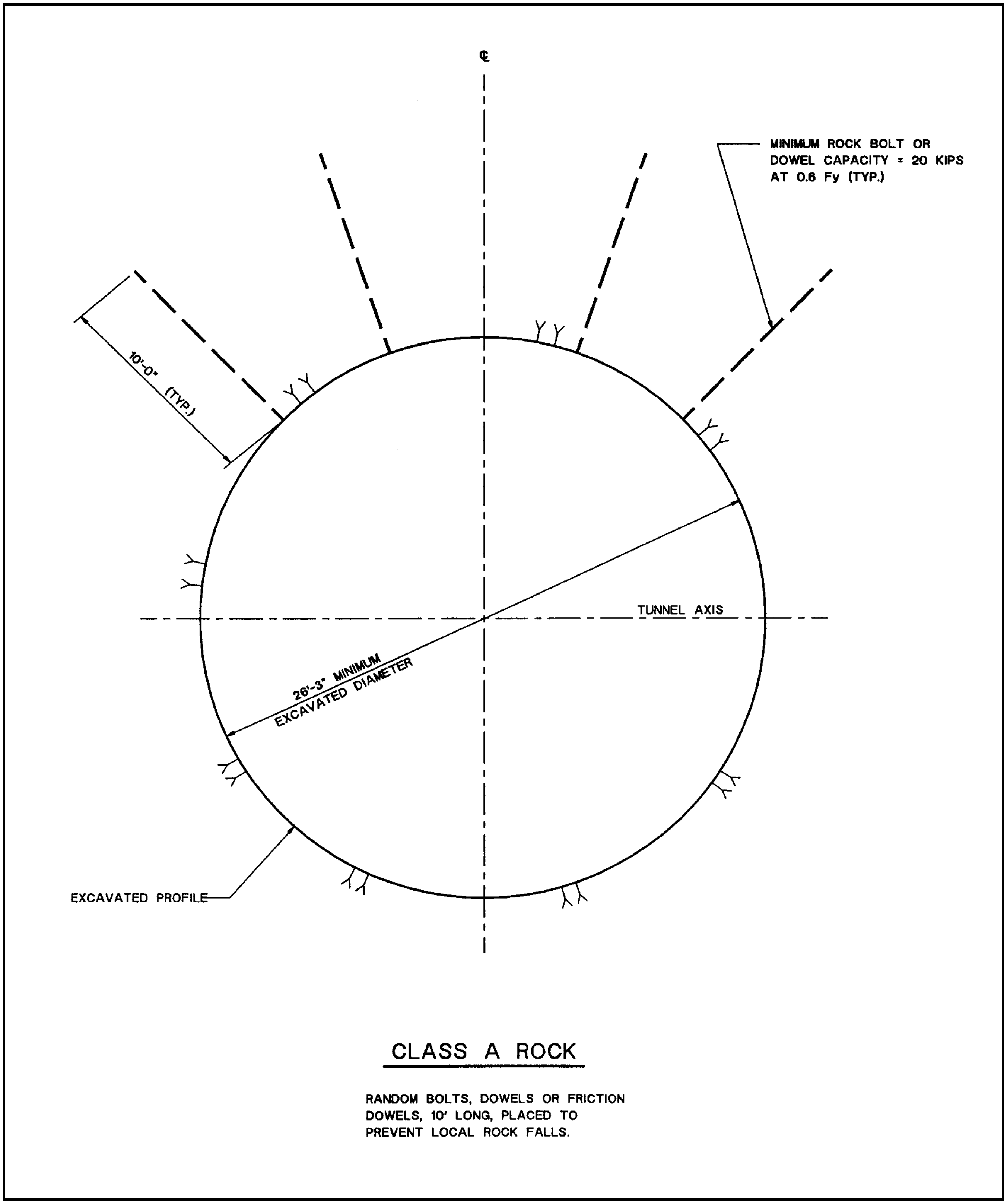



**Figure 5-23. Ground support, Class A rock**

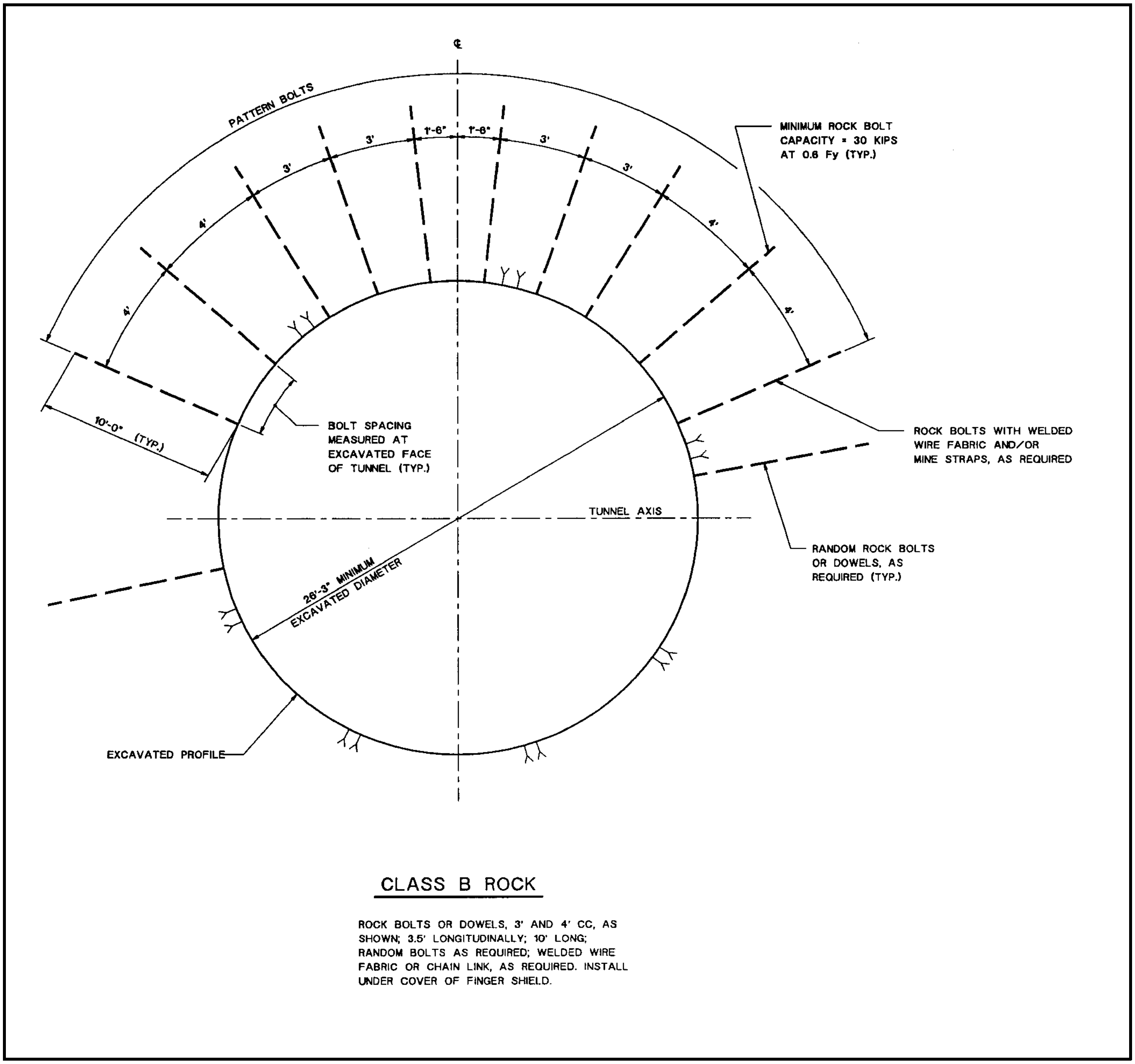



**Figure 5-24. Ground support, Class B rock**

*e*. The types of portal treatments that may be considered include the following:

- No support at the portal when excellent geologic conditions prevail.

- Portal canopy only for rock fall protection.

- Rock reinforcement consisting of a combination of rock bolts, steel mesh, shotcrete, and weeps.

- Rock reinforcement and a canopy for very poor conditions.

Tunnel reinforcement is usually more intense in the vicinity of the portal until the effects of the portal excavation are no longer felt.

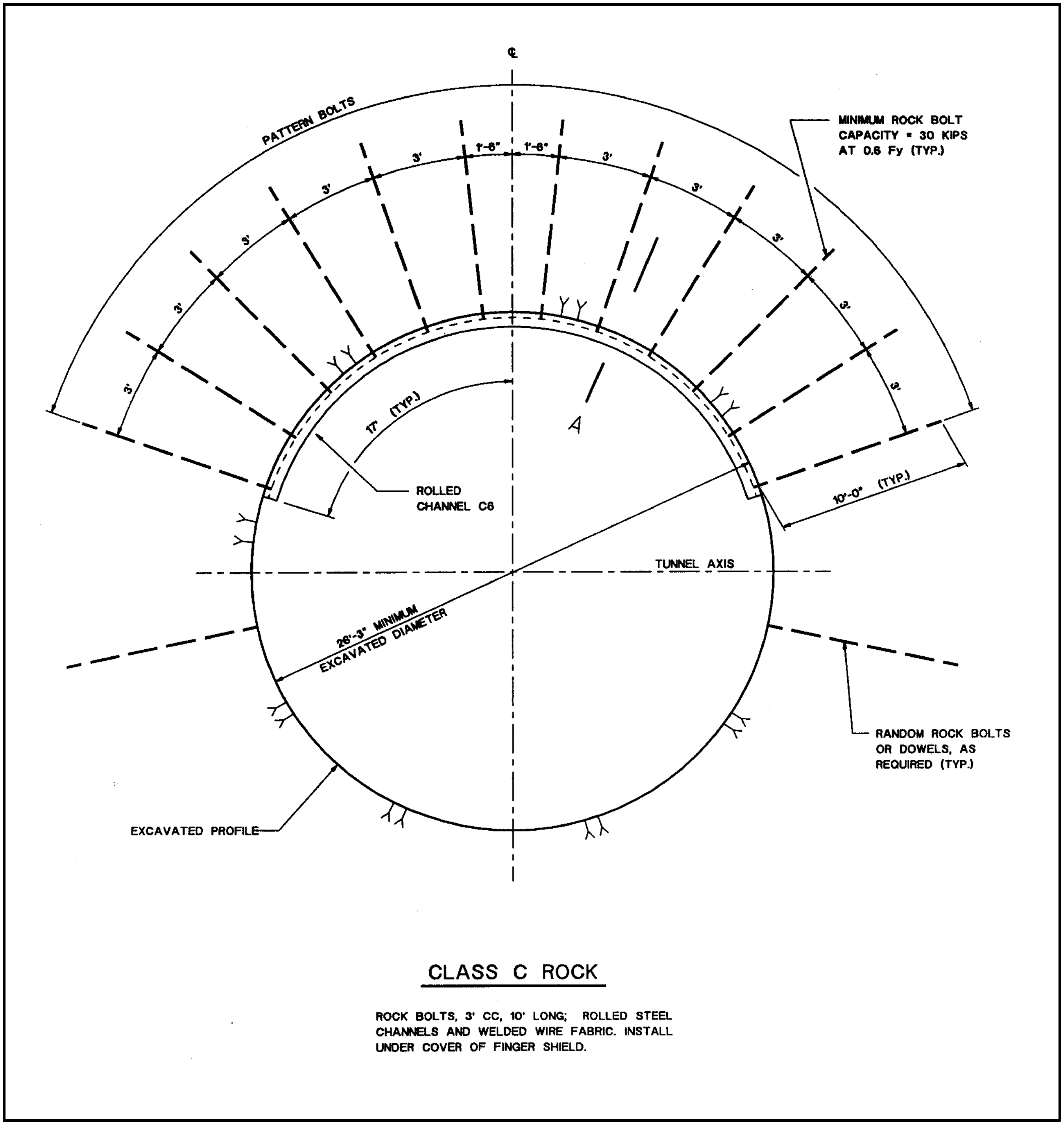



**Figure 5-25. Ground support, Class C rock**

## 5-7. Shaft Construction

Most underground works include at least one deep excavation or shaft for temporary access or as part of the permanent facility. Shafts typically go through a variety of ground conditions, beginning with overburden excavation, weathered rock, and unweathered rock of various types, with increasing groundwater pressure. Shaft construction options are so numerous that it is not possible to cover all of them in this manual. The reader is referred to standard foundation engineering texts for shaft construction,

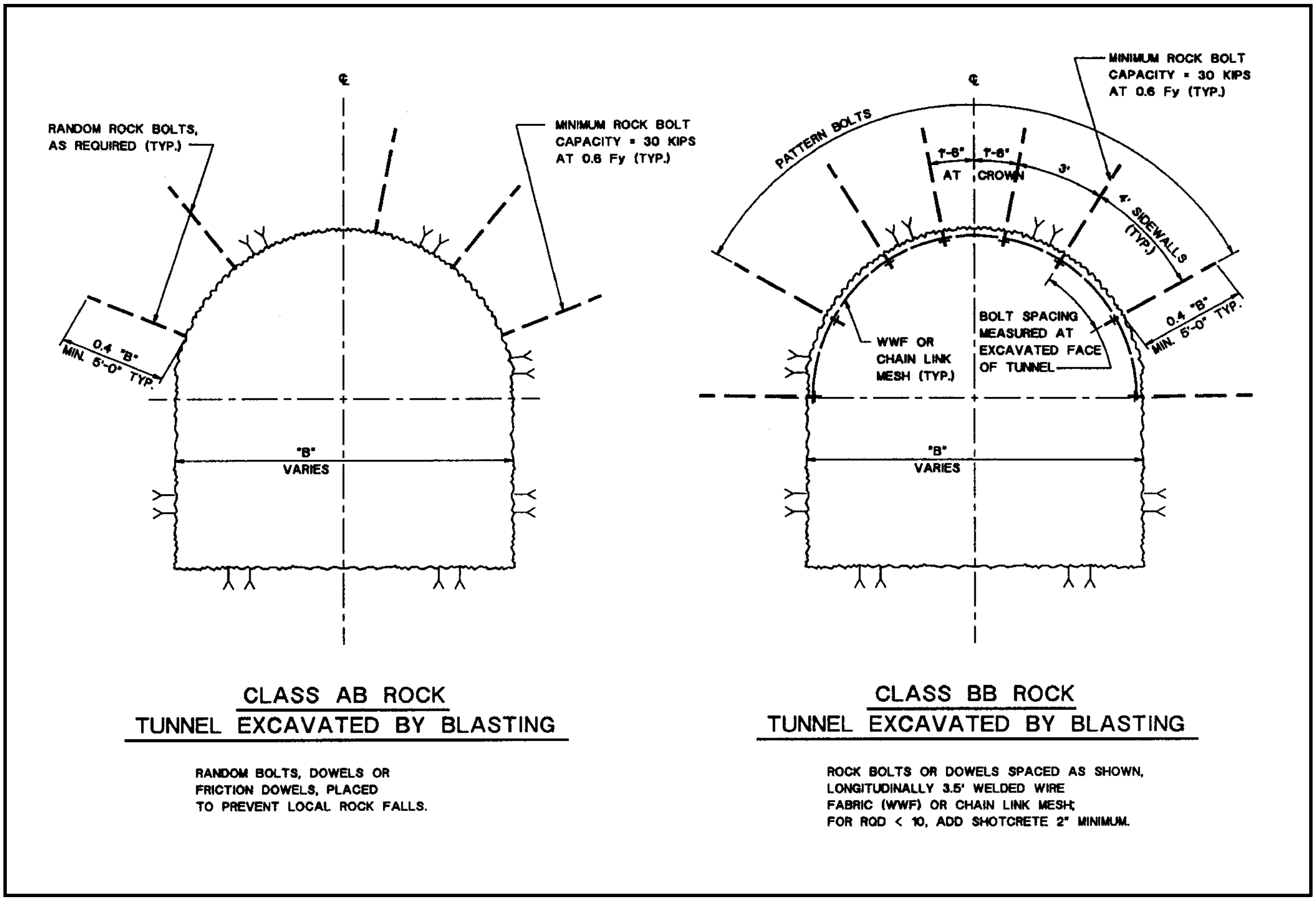



**Figure 5-26. Ground support, blasted tunnel**

temporary and permanent walls through soil and weathered rock, and to the mining literature for deep shafts through rock. The most common methods of shaft excavation and ground support are summarized in this section.

*a. Sizes and shapes of shafts.* Shafts serving permanent functions (personnel access, ventilation or utilities, drop shaft, de-airing, surge chamber, etc.) are sized for their ultimate purpose. If the shafts are used for construction purposes, size may depend on the type of equipment that must use the shaft. Shallow shafts through overburden are often large and rectangular in shape. If space is available, a ramp with a 10-percent grade is often cost-effective. Deeper shafts servicing tunnel construction are most often circular in shape with a diameter as small as possible, considering the services required for the tunnel work (hoisting, mucking, utilities, etc.). Typical diameters are between 5 and 10 m (16-33 ft). If a TBM is used, the shaft must be able to accommodate the largest single component of the TBM, usually the main bearing, which is usually of a size about two-thirds of the TBM diameter.

*b. Shaft excavation and support through soil overburden.*

(1) Large excavations are accomplished using conventional soil excavation methods such as backhoes and dozers, supported by cranes for muck removal. In hard soils and weathered rock, dozers may require rippers to loosen the ground. The excavation size will pose limits to the maneuverability of the excavation equipment.

(2) Smaller shafts in good ground, where groundwater is not a problem, can be excavated using dry drilling methods. Augers and bucket excavators mounted on a kelly, operated by a crane-mounted torque table attachment, can drill holes up to some 75-m (250-ft) depth and 8-m (25-ft) diam. A modified oil derrick, equipped with an elevated substructure and a high-capacity torque table, is also effective for this type of drilling.

(3) Many options are available for initial ground support, including at least the following:

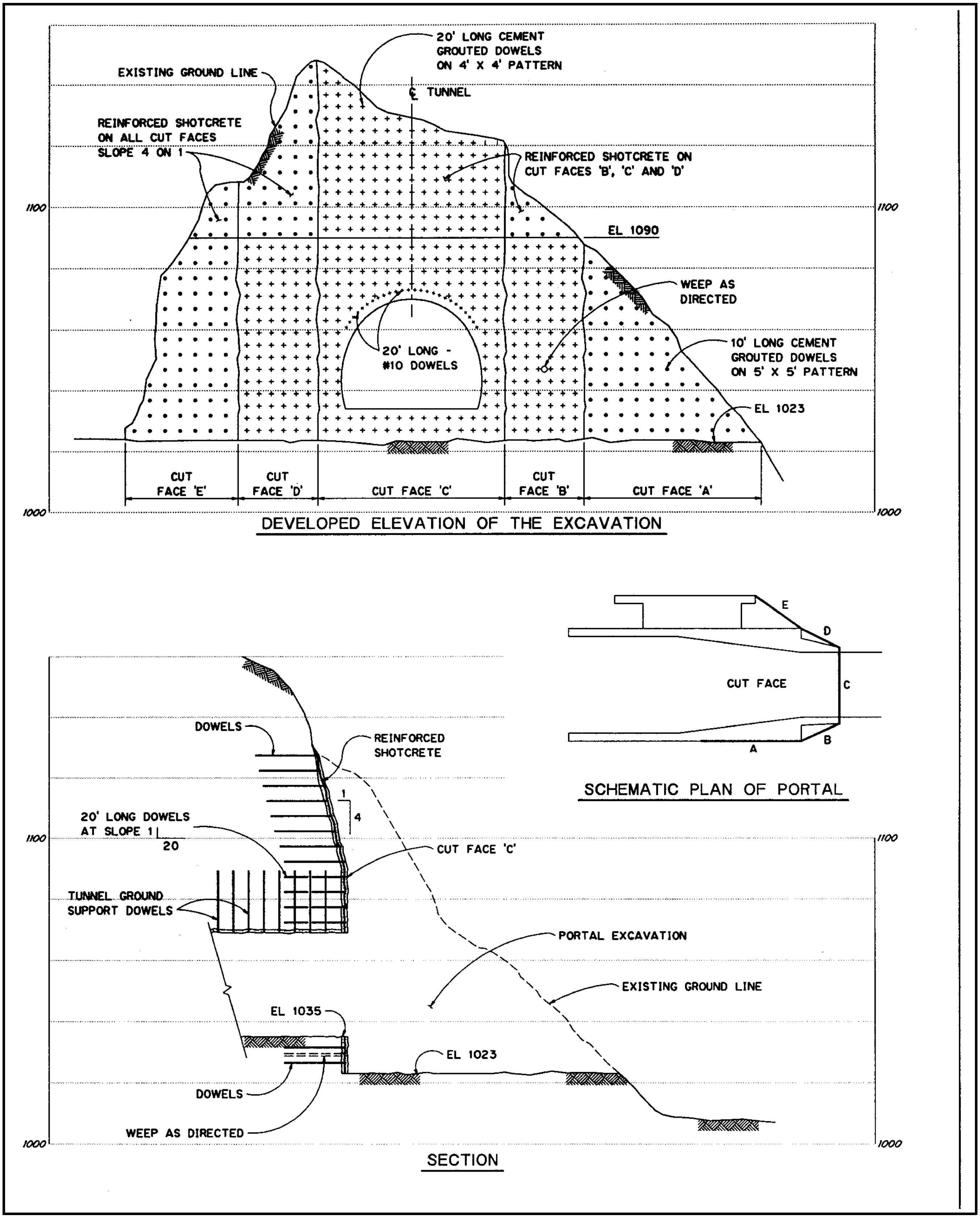



Figure 5-27. Portal excavation and support (H-3 tunnel, Oahu)

- Soldier piles and lagging, in soils where groundwater is not a problem or is controlled by dewatering.

- Ring beams and lagging or liner plate.

- Precast concrete segmental shaft lining.

- Steel sheet pile walls, often used in wet ground that is not too hard for driving the sheet piles.

- Diaphragm walls cast in slurry trenches; generally more expensive but used where they can have a permanent function or where ground settlements and dewatering must be controlled.

- Secant pile walls or soil-mixing walls as substitutes for diaphragm walls, but generally less expensive where they can be used.

(4) Circular shafts made with diaphragm or secant pile walls usually do not require internal bracing or anchor support, provided circularity and continuity of the wall is well controlled. Other walls, whether circular or rectangular, usually require horizontal support, such as ring beams for circular shafts, wales and struts for rectangular shafts, or soil or rock anchors or tiebacks that provide more open space to work conveniently within the shaft.

(5) In good ground above the groundwater table, soil nailing with shotcrete is often a viable ground support alternative.

*c*. *Shaft excavation through rock*.

(1) Dry shaft drilling using a crane attachment or a derrick, as briefly described in the previous subsection, has been proven viable also in rock of strength up to 15 MPa (2,200 psi), provided that the ground is initially stable without support. Use of a bucket with extendable reamer arms permits installation of initial ground support, which would consist of shotcrete and dowels as the shaft is deepened.

(2) Deep shafts can be drilled using wet, reverse circulation drilling. Drilling mud is used to maintain stability of the borehole and counterbalance the formation water pressure, as well as to remove drill cuttings. The drilling is done with a cutterhead, furnished with carbide button cutters and weighted with large donut weights to provide a load on the cutterhead. The drill string is kept in tension, so that the pendulum effect can assist in maintaining verticality of the borehole. Mud is circulated by injecting compressed air inside the drill string; this reduces the density of the drilling mud inside the string and forces mud and drill cuttings up the string, through a swivel, and into a mud pond. From there the mud is reconditioned and led back into the borehole. This type of shaft construction usually requires the installation of a steel lining or casing with external stiffeners, grouted in place. If the steel casing is too heavy to be lowered with the available hoisting gear, it is often floated in with a bottom closure and filled partly with water. This method permits shafts of 2-m (7-ft) diam to be constructed to depths of about 1,000 m (3,300 ft). Larger diameters can be achieved at shallower depths.

(3) If underground access is available, shafts can be drilled using the raise drilling method. A pilot bore is drilled down to the existing underground opening. Then a drill string is lowered, and a drillhead is attached from below. The string is turned under tension using a raise drill at the ground surface, and the shaft is created by backreaming, while cuttings drop into the shaft to the bottom, where they are removed. This method requires stable ground. Raise boring can also be used for nonvertical shafts or inclines. A raised bore can be enlarged using the slashing method of blasting. The bore acts as a large burn cut, permitting blasting with great efficiency and low powder factors.

(4) Conventional shaft sinking using blasting techniques can be used to construct a shaft of virtually any depth, size, and shape. A circular shape is usually preferred, because the circular shape is most favorable for opening stability and lining design. Shaft blasting tends to be more difficult and more confined than tunnel blasting. Typically, shorter rounds are pulled, and the powder factor is greater than for a tunnel in the same material. Variations of the wedge cut are used rather than the burn cut typically used for tunnels. Shallow shaft construction can be serviced with cranes, but deeper shaft construction requires more elaborate equipment. The typical arrangement includes a headframe at the top suspending a two- or three-story stage with working platforms for drilling and blasting, equipment for mucking, initial ground support installation, and shaft lining placement. The typical shaft lining is a cast-in-place concrete lining, placed 10 to 15 m (33-50 ft) above the advancing face.

(5) If the shaft is large enough to accommodate a roadheader, and the rock is not too hard, shaft excavation can be accomplished without explosives using crane service or headframe and stage equipment.

(6) Most shaft construction requires the initial construction of a shaft collar structure that supports overburden and weathered rock near the surface and construction loads adjacent to the top of the shaft. It also serves as a foundation for the temporary headframe used for construction as well as for permanent installations at the top of the shaft.

(7) Inclines of slopes up to about 25 deg can be bored using a TBM specially equipped to maintain its position in the sloping hole. Inclines at any angle can be excavated using blasting methods, with the help of climbing gear such as the Alimak climber.

## 5-8. Options for Ground Improvement

When difficult tunnel or shaft construction conditions are foreseen, ground improvements are often advisable and sometimes necessary. There are, generally speaking, three types of ground improvement that can be feasibly employed for underground works in rock formations:

- Dewatering.
- Grouting.
- Freezing.

*a. Ground improvement for shaft sinking.*

(1) Ground improvement must be considered when shaft sinking involves unstable ground associated with significant groundwater inflow. At a shallow depth, groundwater is often found in potentially unstable, granular materials, frequently just above the top of rock. If sufficiently shallow, the best solution is to extend the shaft collar, consisting of a nominally tight wall, into the top of rock. Shallow groundwater can also often be controlled by dewatering.

(2) An exploratory borehole should be drilled at or close to the center of all shafts. Borehole permeability (packer) tests can be used to estimate the potential groundwater inflow during construction that could occur if the groundwater were not controlled. If the estimated inflow is excessive, ground improvement is called for. At the same time, core samples will give an indication of ground stability as affected by groundwater inflow. Poorly cemented granular sediments and shatter zones are signs of potential instability.

(3) Deep groundwater usually cannot be controlled by dewatering; however, grouting or freezing can be tried. This is usually done from the ground surface before shaft sinking commences, because it is very costly to work down the shaft. Both methods require the drilling of boreholes for the installation of freeze pipes or for grouting. When the shaft is very deep, high-precision drilling is required to reduce the deviation of boreholes to acceptable magnitudes. Considering that borehole spacings are of the order of 1.5 to 2 m (6-7 ft) and that both grouting and freezing rely on accurate placement of the holes, it is readily appreciated that even a deviation of 1 m can be critical. Nonetheless, freezing and grouting have been successfully carried out to depths greater than 500 m (1,700 ft). It is also readily appreciated that both grouting and freezing are very costly; however, they are often the only solutions to a serious potential problem.

(4) Freezing is often more expensive than grouting, and it takes some time to establish a reliable freeze wall, while grouting can be performed more quickly. Professionals in the shaft sinking business generally consider freezing to be substantially more reliable and effective than grouting. It is not possible to obtain a perfect grout job—a substantial reduction of permeability (say, 80-90 percent) is the best that can be hoped for—and grouting may leave some areas ungrouted. On the other hand, a freeze job can more readily be verified and is more likely to create a continuous frozen structure, thus is potentially more reliable.

(a) *Grouting*. General advice and design recommendations for grouting are found, for example, in EM 1110-2-3506, Grouting Technology, and in Association Française (1991). The detailed grouting design for deep shafts is often left to a specialist contractor to perform and implement. While chemical grouting is often used in loose sediments and overburden materials, grouting in rock is usually with cement. Grout penetration into fractures is limited by aperture of the fractures relative to the cement particle sizes. As a rule, if the rock formation is too tight to grout, it is also usually tight enough that groundwater flow is not a problem. Shaft grouting typically starts with the drilling of two or three rows of grout holes around the shaft perimeter, spaced 1.5 to 2.0 m (5-7 ft) apart. Grout injection is performed in the required zones usually from the bottom up, using packers. The effectiveness of the grout job can be verified by judicious sequencing of drilling and grouting. If secondary grout holes drilled after the first series of grouted holes display little or no grout take, this is a sign of the effectiveness of grouting. Additional grout holes can be drilled and grouted as required, until results are satisfactory. If it becomes necessary to grout from the bottom of the shaft, indicated, for example, by probeholes drilled ahead of the advancing shaft, then grout holes are drilled in a fan pattern covering the stratum to be grouted. It is important to perform the grouting before a condition has arisen with large inflows, because grouting of fissures with rapidly flowing water is very difficult. When drilling from the bottom of a deep shaft, it is often necessary to drill through packers or stuffing boxes to prevent high-pressure water from entering the shaft through the drillholes.

(b) *Freezing*. Brine is usually used as the agent to withdraw caloric energy from the ground and freeze the water in the ground. The brine is circulated from the refrigeration plant in tubes placed in holes drilled through the ground to be frozen. The tubes can be insulated through ground that is not intended to be frozen. The detailed design and execution of a freezing program requires specialist knowledge and experience that is only available from firms that specialize in this type of work. The designer of the underground work should prepare a performance specification and leave the rest to the contractor and his specialist subcontractor. The detailed design of a freeze job includes the complete layout of plant and all freeze pipes so as to achieve a freeze wall of adequate strength and thickness and thermal analyses to estimate the required energy consumption and the time required to achieve the required results, with appropriate safety factors. The English-language literature does not offer a great number of references on ground freezing. One source is the Proceedings of the Third International Symposium on Ground Freezing (USACE 1982). The strength of frozen ground is dependent on the character and water content of the ground and increases with decreasing temperature of the frozen ground. Some rock types, notably weak, fine-grained rocks, suffer a substantial strength loss upon thawing. The effects of thawing must be considered in the design of the final shaft lining. Saline groundwater is more difficult to freeze because of its lower freezing temperature. If the formation water is not stagnant but moves at an appreciable rate, it will supply new caloric energy and delay the completion of the freeze job. The velocity of formation water movement should be estimated ahead of time, based on available head and gradient data. At the ground surface, brine distribution pipes are often laid in a covered trench or gallery around the shaft, keeping them out of the way from shaft construction activities. Since freezing involves expansion of the formation water, a relief borehole is usually provided at the center of the shaft so that displaced water can escape. The freezing process is controlled by installing temperature gages at appropriate locations between freeze pipes, as well as through monitoring of the temperature of return brine and the overall energy consumption. On rare occasions it becomes necessary to implement a freezing installation from the bottom of the shaft. This usually requires the construction of a freezing gallery encircling the shaft. Shaft excavation cannot proceed during the implementation of an underground freeze job, including the time required to achieve the necessary reduction in ground temperature. Down-the-shaft freezing, therefore, is very costly. Quicker implementation of a freezing application can be accomplished using liquid nitrogen as coolant rather than brine.

*b*. *Ground improvement for tunneling*. Rock tunnels generally do not require ground improvement as frequently as shafts. Examples of ground improvements using grout applications are briefly described in the following.

(1) *Preconstruction application*. Where it is known that the tunnel will traverse weak ground, such as unconsolidated or poorly consolidated ground or a wide shatter zone, with high water pressure, the ground can be grouted ahead of time. It is preferable to grout from the ground surface, if possible, to avoid delaying tunneling operations. Such grout applications are particularly helpful if the water is contaminated with pollutants or if the groundwater is hot. The primary purpose of applying grout is to reduce the ground's permeability. Strengthening of the ground is sometimes a side benefit.

(2) *Application during construction*. When grouting cannot be applied from the ground surface, it can be carried out from the face of the tunnel before the tunnel reaches the region with the adverse condition. An arrangement of grout holes are drilled in fan shape some 20 to 40 m (60-130 ft) ahead of the face. Quality control is achieved by drilling probeholes and testing the reduction of permeability. Grouting is continued until a satisfactory permeability reduction is achieved.

(3) *Application after probehole drilling*. Where adverse conditions are expected but their location is unknown, probehole drilling will help determine their location and characteristics. Such probeholes can be simple percussion holes with a record of water inflow, or packer tests can be performed in these probeholes. The grout application can be designed based on the results of one or more probeholes.

(4) *Postexcavation grouting*. If it is found that water inflow into the excavated tunnel is too large for convenient placement of the final lining, radial grouting can be performed to reduce the inflow. Generally, the grout is first injected some distance from the tunnel, where water flow velocities are likely to be smaller than at closer distances. It is sometimes necessary to perform radial grouting after the completion of the tunnel lining. Here, the finished

lining helps to confine the grout, but the lining must be designed to resist the grout pressures.

(5) *Freezing in tunnels*. Freezing is sometimes a suitable alternative to grouting for temporary ground strengthening and inflow control. Freezing is particularly effective if the ground is weak, yet too impervious for effective grout penetration.

## 5-9. Drainage and Control of Groundwater

*a*. *General*. The design of a permanent drainage system and the control systems required for groundwater begins during the geotechnical exploration phases with an assessment of the potential sources and volumes of water expected during construction. The type of permanent drainage system required will depend upon the type of tunnel and site groundwater conditions.

*b*. *Assessment of water control requirements*. Prior to construction, estimates of the expected sources of groundwater and the expected inflow rates and volumes must be identified in order for the contractor to provide adequate facility for handling inflow volumes. Section 3-5 provides guidance in identifying potential sources of groundwater and for making inflow volume estimates.

*c*. *Care of groundwater during construction*.

(1) Care of groundwater generally is the responsibility of the contractor; however, the specifications for a tunnel contract may require that certain procedures be followed. For example, if it is expected that water-bearing joints will be present that contain sufficient head and volume to endanger the safety of the tunnel, the drilling of a probehole ahead of the working face should be required. The following discussion is for guidance.

(2) Water occurring in a tunnel during construction must be disposed of because it is a nuisance to workers and may make the placement of linings difficult or cause early weakening of the linings. It also makes the rock more susceptible to fallout by reducing the natural cohesion of fine-grained constituents.

(3) The excavation sequence should be such that drainage of the sections to be excavated is accomplished before excavation. Thus, a pilot drift near the invert in a wet environment is more effective than a top heading although enlargement to full size is more difficult. It is an excellent practice to carry a drill hole three tunnel diameters in advance of the working face. The drill hole has an additional advantage of revealing rock conditions more clearly than defined by the initial investigation.

(4) When encountered, water should be channeled to minimize its effect on the remaining work. To accomplish this, the surface of a fissure may be packed with quick-setting mortar around a tube leading to a channel in the invert. Ingenuity on the part of workers and supervisors can produce quick, effective action and should be encouraged so long as objectionable materials do not intrude within the concrete design line.

(5) If groundwater inflow is extremely heavy and drainage cannot be accomplished effectively, it will be necessary to install a "grout umbrella" from the face before each tunnel advance is made. This consists of a series of holes angled forward and outward around the perimeter of the face that are pumped with grout to fill fractures and form a tunnel barrier against high inflows.

(6) For permanent protection from the flow of water along the outside of the concrete lining, no better method exists than filling with grout any void that remains after the concrete is set.

(7) Section 5-14.b. provides additional information on the control and disposal of groundwater.

*d*. *Permanent drainage systems*.

(1) *Drainage system*. The drainage system required in a tunnel will depend on the type of tunnel, its depth, and groundwater conditions. Some tunnels may not require special drainage. Others may require drainage to limit the pressure behind the lining or to remove water due to condensation and leakage through the tunnel joints. A detailed design procedure for drains will not be attempted here; however, a brief description will be included to indicate what is involved in providing drainage for the various types of tunnels.

(a) *Pressure tunnels*. Drainage for pressure tunnels may be required if normal outlets through gates or power units do not accomplish complete unwatering of the tunnel. The drains are then located at the low point of the tunnel and are provided with a shutoff valve. In some cases, it is desirable to provide drainage around a pressure tunnel. This may be done to limit the external head on the lining or to limit pressures in a slope in the event leakage developed through the lining. Drainage may be provided by drilling holes from the downstream portal or by a separate drainage tunnel.

(b) *Outlet tunnels.* Drainage for outlet tunnels may be required to completely unwater the tunnel if some point along the tunnel is lower than the outlet end. To limit the external head, drains can be provided that lead directly into the tunnel. In this manner, the outlet tunnel also serves as a drain tunnel.

(c) *Vehicular tunnels*. Drainage for vehicular tunnels will usually consist of weep holes to limit the pressure behind the lining and an interior drain system to collect water from condensation and leakage through the joints in the lining. Interior drainage can be either located in the center of the tunnel between vehicular wheel tracks or along the curbs. If the tunnel is located in areas where freezing temperatures occur during part of the year, precautions should be taken to prevent freezing of the drains. If the tunnel is long, protection against freezing need not be installed along the entire length of tunnel, depending on the climate and depth at which the tunnel is located.

(d) *Drain and access tunnels*. Drainage from these tunnels may require a sump and pump, depending on the location of the outlet end. Drain tunnels usually have drain holes that extend from the tunnel through the strata to be drained.

(e) *Waterstop.* To prevent uncontrolled water seepage into a concrete-lined tunnel, the construction joints are waterstopped. EM 1110-2-2102 covers the types and use of waterstops.

(2) *Grouting*. Grouting in connection with tunnel construction is covered in paragraph 28 and Plate 5 of EM 1110-2-3506. Recommendations are made below regarding special grouting treatment typically required to prevent drainage problems in various types of tunnels or shafts. Ring grouting (i.e., grouting through radial holes drilled into the rock at intervals around the tunnel periphery) is used to reduce the possibility of water percolating from the reservoir along the tunnel bore and for consolidation grouting along pressure tunnels. Contact grouting refers to the filling of voids between concrete and rock surface with grout.

(a) *Outlet works tunnels*. As a minimum, the crown of outlet works tunnels should be contact grouted for their entire length. Grouting to prevent water from percolating along the tunnel bore should consist of a minimum of one ring, interlocked with the embankment grout curtain. If the impervious core of the embankment extends upstream from the grout curtain and sufficient impervious material is available between the tunnel and the base of the embankment, the location near the upstream edge of the impervious core also should extend into the rock approximately one tunnel diameter.

(b) *Pressure tunnels*. Pressure tunnel linings are designed in two ways. Either the concrete and steel linings act together to resist the entire internal pressure or concrete and steel linings and the surrounding rock act together to resist the internal pressure. Contact and ring grouting for pressure tunnels is done the same as for outlet tunnels except one additional ring should be grouted at the upstream end of the steel liner. Consolidation grouting of the rock around the lining of a pressure tunnel and the filling of all voids is a necessity if the rock is to take part of the radial load. Consolidation grouting of the rock behind the steel liner is good practice and should be done whether or not the rock is assumed to resist a portion of the internal pressure.

(c) *Shafts*. Shafts are normally grouted the same as tunnels except that grouting is done completely around the shaft in all cases.

## 5-10. Construction of Final, Permanent Tunnel Linings

When the initial ground support components described in the previous sections do not fulfill the long-term functional requirements for the tunnel, a final lining is installed. On occasion, an initial ground support consisting of precast segments will also serve as the final lining (see Section 5-4.i). More typically, the final lining will be constructed of cast-in-place concrete, reinforced or unreinforced, or a steel lining surrounded by concrete or grout. Guidelines for the selection of a final lining is presented in Section 9-1. The following subsections describe cast-in-place concrete lining and steel lining construction.

*a. Cast-in-place concrete lining*. When a concrete lining is required, the type most commonly used is the cast-in-place lining. This lining provides a hydraulically smooth inside surface, is relatively watertight, and is usually cost competitive. Concrete linings can be of the following types:

- Unreinforced concrete.
- Concrete reinforced with one layer of steel, largely for crack control.
- Concrete reinforced with two layers of steel, for crack control and bending stresses.

- Unreinforced or reinforced concrete over full waterproofing membrane.

(1) *Placement sequence*. Depending on tunnel size and other factors, the entire cross section is placed at one time, or the invert is placed first, or the invert is placed last. Sometimes precast segments are placed in the invert to protect a sensitive rock from the effects of tunnel traffic, followed by placement of the crown concrete. This method will leave joints between the invert segments, but these joints can be designed for sealing or caulking. Barring construction logistics constraints, the most efficient method of placement is the full-circle concreting operation. When schedule or other constraints require that concrete be placed simultaneously with tunnel excavation and muck removal through the tunnel segment being concreted, then either the precast-invert segment method or the arch-first method is appropriate. Depending on the tunnel size, the upper 270 deg of a circular tunnel are placed first to permit construction traffic to flow uninterrupted and concurrently with lining placement. With the precast-invert segment method, the segment is made wide enough to permit all traffic operations. The invert-first placement method is not now frequently used for circular tunnels, in part because the invert takes time to cure and is subject to damage during placement of the crown. This method is sometimes advantageous when a waterproofing membrane is used. When the final lining is horseshoe-shaped, the invert is usually placed first, furnished with curbs to guide the placement of sidewall forms. Sometimes, especially in tunnels with ribs as initial ground support, L-shaped wall foundations are placed first; these will then guide the placement of the invert and the side walls.

(2) *Formwork*. Except for special shapes at turns and intersections, steel forms are used exclusively for tunnels of all sizes. The forms often come in widths of 1.5 to 1.8 m (5-6 ft), with provisions to add curve filler pieces to accommodate alignment radii. The segments are hinged and collapsible to permit stripping, transporting, and reerection, using special form carriers that ride on rails or rubber tires. The forms are usually equipped with external vibrators along with provisions to use internal vibrators through the inspection ports if necessary. Telescoping forms permit leapfrogging of forms for virtually continuous concrete placement.

(3) *Concrete placement*. Placement is accomplished using either of two methods: the conventional slick line method and the multiport injection method.

(a) The slick line is a concrete placement pipe, 150 to 200 mm (6-8 in.) in size, placed in the crown from the open end of the form up to the previously placed concrete. Concrete is pumped into the form space until a sloping face of the fresh concrete is created in the form space. The slick line is gradually withdrawn, keeping the end of the pipe within the advancing fresh concrete. Minimum depth of pipe burial varies between 1 and 3 m (3-10 ft), depending on size of tunnel and thickness of lining. The advancement of the concrete is monitored through inspection ports and vibrated using form vibrators and internal vibrators.

(b) With the injection method, special injection ports are built into the form, through which concrete is placed using portable pumping equipment. Again, placement occurs in the direction from the previously placed concrete. Depending on the diameter of the tunnel, one to five injection ports may be located at any given cross section, with one port always at the crown. For large-diameter tunnels, and for reinforced linings, it is inadvisable to let the fresh concrete fall from the crown to the invert. Here, concrete must be placed through ports. Concrete forms are usually stripped within 12 hr of placement so as to permit placement of a full form length every day. Concrete must have achieved enough strength at this time to be self-supporting. Usually a strength of about 8.3 MPa (1,200 psi) is sufficient.

(4) *Groundwater control during concreting*. Water seepage into the tunnel may damage fresh concrete before it sets. Side wall flow guides, piping, and invert drains may be used to control water temporarily. After completion of the lining, such drain facilities should be grouted tight. High-water flows may require damming or pumping, or both, to remove water before placing concrete. On occasion, formation grouting may be required.

(5) *Concrete conveyance*. The concrete is brought from the surface to the tunnel level either by pumping or through a drop pipe. If conveyed through a drop pipe, the concrete is remixed to eliminate separation. If the concrete is pumped, the pumping may continue through the tunnel all the way to the point of placement. Depending on the distance, booster pumps may be used. If possible, additional shafts are placed along the tunnel to reduce the distance of concrete conveyance in the tunnel. Other conveyance methods in the tunnel include conveyors, agitator cars, or nonagitated cars, trammed by locomotives to the point of placement. Remixing may be required, depending on the system used, to maintain the proper consistency of the fresh concrete. It is also possible at this location to add an accelerator if necessary. When conveying concrete for long distances, it is possible to add a retarder to

maintain fluidity, then supplemented with an accelerator prior to placement.

(6) *Construction joints*. Transverse joints are located between pours, often 30 m (100 ft) apart or up to nearly 60 m (200 ft), depending on the form length used by the contractor. Either a sloping joint or a vertical joint can be used. Either type will result in a structurally acceptable joint. When a sloping joint is used, a low bulkhead is usually used to limit the feathering out of the concrete at the invert. The advantage of the sloping joint is that only a low bulkhead is required; this method is least likely to result in voids when using a slick line method. Disadvantages of the sloping joint include the following:

- Difficulty in proper preparation of joints before the next pour.

- Waterstop placement not feasible.

- Underutilization of total length of the form.

- Formation of much longer construction joint, compared with the vertical joint.

The sloping joint is often more convenient when an unreinforced lining is constructed. The advantages of the vertical joint are accessibility of the joints for proper preparation, formation of the shortest possible length of joint, and full utilization of formwork. The vertical joint is most often used with reinforced concrete linings. Some of the disadvantages include the additional time required for bulkhead installation, provisions for maintaining reinforcing steel continuity across the joint, and the probability of forming voids when using the slick line method. From the perspective of watertightness, longitudinal joints resulting from the two-pour methods are not desirable. In particular, the arch-first method poses the greatest difficulty in joint surface treatment to achieve desired watertightness. Waterstops are not used for construction joints in unreinforced concrete linings. Water stops and expansion joints are of doubtful value in reinforced concrete linings but are sometimes used at special locations, such as at changes in shape of opening, intersections, and transitions to steel-lined tunnels.

(7) *Contact grouting*. When a tunnel lining has to withstand appreciable loads, external or internal, it is essential that the lining acts uniformly with the surrounding rock mass, providing uniformity of loading and ground reaction. Hence, significant voids cannot be tolerated. Voids are often the result of imperfect concrete placement in the crown. Voids are virtually unavoidable in blasted tunnels with irregular overbreak. It is therefore standard practice to perform contact grouting in the crown, using groutholes that have been either preplaced or drilled through the finished lining, so as to fill any crown voids that remain. Grouting is usually made to cover the upper 120 to 180 deg of circumference, depending on tunnel size and amount of overbreak. USACE has a guide specification for Tunnel and Shaft Grouting, available from HQUSACE.

(8) *Supplementary grouting and repair*. In the event that groundwater leaks excessively into the finished tunnel, formation grouting can be used to tighten the ground. This is done through radial groutholes through the lining. Leaking joints can also be repaired by grouting or epoxy treatment.

*b*. *Steel lining*. A steel lining is required when leakage through a cracked concrete lining can result in hydrofracturing of the surrounding rock mass or deleterious leakage or water loss. In most respects, the steel lining is similar to open-air penstocks, except that the tunnel steel lining is usually designed for an exterior water pressure and is furnished with external stiffeners for high external pressure conditions. Fabrication and assembly of a steel lining generally follow the same standards and practices as penstocks described in American Society of Civil Engineers (ASCE) (1993). Some construction aspects of steel-lined tunnels, however, deserve special attention, particularly as they affect the preparation of contract documents.

(1) *Constructibility*. Individual pipes and joints are usually made as large as can be practically transported on the highway to the site and into the tunnel for placement and joining, leaving field welding to a minimum. Each motion through shafts, adits, and tunnel must be considered in the evaluation of the maximum size of the individual pieces.

(2) *Handling and support*. Pipes without external stiffeners should be internally supported during transport and installation if their diameter/thickness ratio, D/t, is less than 120. The internal bracing can be timber or steel stulling (see ASCE 1993) or spiders with adjustable rods. The minimum thickness of the steel shell is usually taken as $t_{min} = (D + 20)/400$, with dimensions in inches, or more simply $t_{min} = D/350$ (in inches or millimeters). Externally coated pipes must be protected from damage to coating, using appropriate support and handling, e.g., fabric slings.

(3) *Support during concrete placement.* The pipe must be centrally aligned in the excavated tunnel and prevented from distortion and motion during concrete placement. This may require the pipe to be placed on cradles, usually of concrete, with tiedowns to hold the pipe in place against flotation and internal stulling. Steel or concrete blocking (not timber) is often used to resist flotation.

(4) *Jointing.* Welding procedures, including testing of welds, are similar to those of surface penstocks. It is often impractical to access the exterior of the pipe for welding and testing. An external backup ring, though less satisfactory, may be required. All welds should be tested using nondestructive testing methods using standards of acceptance similar to surface penstocks (see ASCE 1993).

(5) *Concrete placement*. The tunnel must be properly prepared for concrete placement. Because the concrete must provide a firm contact between steel and ground, all loose rock and deleterious materials, including wood blocking, must be removed and groundwater inflow controlled as discussed in the previous subsection. Adequate clearances must be provided around the pipe. The concrete is usually placed using the slick line method. The concrete mix should be selected to minimize the buildup of heat due to hydration; subsequent cooling will result in the creation of a thin void around the pipe. Usually a relatively low strength (14 MPa, 2,000 psi, at 28 days) is adequate. Sloping cold joints are usually permissible.

(6) *Contact grouting.* Grouting applications include the filling of all voids between concrete backfill and rock, which is termed contact grouting, and skin grouting of the thin void between steel lining and concrete. Contact grouting is often carried out through grout plugs provided in the pipe, located at the top and down 15 and 60 deg on each side to cover the upper 180 deg of installation. The grout plugs are spaced longitudinally every 3 m (10 ft), staggered, or between stiffeners if the pipe has external stiffeners. Grout holes are drilled through the predrilled holes in the steel plate, the concrete, and up to about 600 mm (2 ft) into the surrounding rock. The grout is a sand-cement mix, applied at pressures up to 0.7 MPa (100 psi).

(7) *Skin grouting.* The purpose of skin grouting is to fill the thin void that may exist between concrete and steel after the concrete cures. Theoretically, skin grouting is not required if a conservative value of the void thickness has been assumed in design, and a safe and economical structure can be achieved without skin grouting. If skin grouting is to be performed, it is usually according to the following procedure:

(a) After curing of the concrete (days or weeks), sound the steel for apparent voids and mark the voids on the steel surface.

(b) Drill 12- to 18-mm (0.5- to 0.75-in.) holes at the lower and the upper part of the voids.

(c) Grout with a flowable nonshrink grout, using the upper hole as a vent.

(d) After grout has set, plug holes with threaded plugs and cap with a welded stainless steel plate.

## 5-11. Ventilation of Tunnels and Shafts

Shaft and tunnel construction generally occurs in closed, dead-end spaces, and forced ventilation is essential to the safety of the works. Specifically, the Occupational Health and Safety Act (OSHA) 10 CFR 1926 applies to construction work; Subpart S, CFR 1926.800, applies to underground construction. USACE's EM 385-1-1, Safety and Health Requirements Manual, also applies. Some states have regulations that are more stringent than Federal regulations (see the California Tunnel Safety Orders). Contractors are responsible for the safety of the work, including temporary installations such as ventilation facilities and their operation and are therefore obliged to follow the law as enforced by OSHA. Contract documents do not usually contain specific requirements for ventilation, because such specific requirements might be seen as overriding applicable laws. In special cases, however, the tunnel designer may choose to incorporate specific ventilation requirements, supplementary to the applicable regulations. In such cases, the purpose is to make sure that the contractor is aware of the specific circumstances. By requesting submittals from the contractor on ventilation items, the owner/engineer can ascertain that the contractor does, indeed, follow regulations. Circumstances that may call for ventilation specification requirements include the following:

- An unusually long tunnel without intermediate ventilation shaft options.

- Certain potentially hazardous conditions, such as noxious or explosive gas occurrences, hot water inflow.

- Particularly extreme environmental conditions, such as very hot or very cold climatic conditions, where heating or cooling of air may be required.

- Circumstances where the ventilation system is left in place for use by a subsequent contractor or the owner; in these cases, the ventilation system should be designed almost as a part of the permanent system, rather than a temporary installation.

*a*. *Purposes of underground ventilation*. Underground ventilation serves at least the following purposes:

- Supply of adequate quality air for workers.

- Dilution or removal of construction-generated fumes from equipment and blasting or of gases entering the tunnel.

- Cooling of air—heat sources include equipment, high temperature of in situ rock or groundwater, high ambient temperature.

- Heating of air—sometimes required to prevent creation of ice from seepage water or from saturated exhaust air.

- Smoke exhaust in the event of underground fire-dust control.

Thus, designers of an underground ventilation system must consider the ambient and in situ temperatures, projected water inflow, potential for adverse conditions (gases), maximum number of personnel in the underground, types and number of equipment working underground, and methods of equipment cooling employed. In the permanent structure, ventilation provisions may be required for at least the following purposes:

- To bleed off air at high points of the alignment.

- To purge air entrained in the water, resulting, for example, from aeration in a drop shaft.

- For odor control and dilution of sulfide fumes in a sewer tunnel.

- To provide ventilation for personnel during inspection of empty tunnels.

These ventilation requirements often result in the use of separate permanent ventilation shafts with appropriate covers and valves.

*b*. *Components of ventilation system*. The principal components of a ventilation system are briefly listed below:

(1) *Fans*. Usually in-line axial or centrifugal fans are used. Fans can be very noisy, and silencers are usually installed. In a sensitive neighborhood, silencers are particularly important; alternatively, fans can be installed a sufficient distance away from the tunnel or shaft portals to reduce noise levels. Fans are designed to deliver a calculated airflow volume at a calculated pressure. With long vent lines, the required pressure may be too high for effective fan operation at one location (air leakage from vent lines also increase with increased differential pressure), and booster fans along the line are used. In the working areas, auxiliary fan installations are often required for dust control, ventilation of ancillary spaces, local air cooling, removal of gases or fumes, or other special services. When auxiliary fan systems are used, such systems shall minimize recirculation and provide ventilation that effectively sweeps the working places. Reversibility of fans is required to permit ventilation control for exhaust of smoke in case of fire.

(2) *Fan lines*. Rigid-wall fan lines made of steel ducting or fiberglass are sometimes used, mostly for exhaust; however, flexible ducting, made of flame retardant material, is more commonly used. Flexible ducting must retain an internal overpressure in order not to collapse. This requires reliable fan start control of all main and booster fans.

(3) *Scrubbers*. Excessive dust is generated from roadheader or TBM operation and is usually exhausted through scrubbers or dust collectors.

(4) *Ancillary ventilation structures*. These may include stoppings and brattices to isolate areas with different ventilation requirements or where no ventilation is required. In hot environments, cooling can be applied to the entire ventilation system, or spot coolers can be applied to working areas. Heaters can be required to prevent ice from forming at exhausts.

(5) *Monitors and controls*. These include air pressure and air flow monitors within the ducting or outside, monitoring of gases (methane, oxygen, carbon monoxide, radon, and others), temperature, humidity, and fan operation status. Stationary gas detectors located at strategic points in the ventilation system and at the face (e.g., mounted on the TBM) are often supplemented with hand-held detectors or sampling bottles. Signals would be monitored at the ventilation control center, usually at the ground surface, where all ventilation controls would be operated. Secondary monitors are often installed at the working area underground.

*c*. *Design criteria*. Typically, an air supply of at least 2.83 $m^3$/min (100 cfm) per brake horsepower of installed diesel equipment is required. Gasoline-operated equipment is not permitted, and diesel equipment must be provided with scrubbers and approved for underground operation. Mobile diesel-powered equipment used underground in atmospheres other than gassy operations shall be approved by MSHA (30 CFR Part 32), or shall be demonstrated to meet MSHA requirements. An additional air supply of 5.7 $m^3$/min (200 cfm) is required for each worker underground. Ventilation should achieve a working environment of less than 27 °C (80 °F) effective temperature, as defined in Hartman, Mutmansky, and Wang (1982). A minimum air velocity in the tunnel of 0.15 m/s (30 fpm) is usually required, but 0.5 m/s (100 fpm) is desirable. Air velocity should not exceed 3 m/s (600 fpm) to minimize airborne dust. For additional design criteria and methods, see SME Mining Engineering Handbook (1992) and ASHRAE Handbook (1989).

## 5-12. Surveying for Tunnels and Shafts

Technological advances in survey engineering have had a great influence on the design and construction of tunnels and shafts. From initial planning and integration of geotechnical and geographical data with topographical and utility mapping through the actual alignment and guidance of tunnel and shaft construction, survey engineering now plays a major role in the overall engineering and construction of underground structures. To benefit from these advances, survey engineers should be involved from the inception of planning through design and final construction. The results of these surveys would provide more cost-effective existing-conditions data, ranging from topographic mapping to detailed urban utility surveys; the use of appropriate coordinate systems tailored to meet the specific needs of the project; optimized alignments; more accurate surface and subsurface horizontal and vertical control networks properly tied to other systems and structures; precise layout and alignment of shaft and tunnel structures; and significant reduction in the impact of survey operations on tunnel advance rates.

*a*. *Surveying and mapping tasks during planning*.

(1) During the planning stage, the framework is constructed for all future project surveying and mapping efforts. Among the many important tasks to be performed at an early stage are the following:

- Select basic coordinate system and horizontal and vertical datums.
- Select or develop project-specific coordinate and mapping system.
- Provide tie-in with existing relevant coordinate and datum systems.
- Verify or renew existing monumentation and benchmarks.
- Develop specifications for required surveying and mapping activities.
- Procure existing map base and air photos as required.
- Supplement mapping as required for the purpose of planning.
- Prepare a Geographic Information System (GIS) base for future compilation of site data.

(2) In the United States, the standard reference for surveying is the North American Datum 1983 (NAD'83) for horizontal datum, and the North American Vertical Datum of 1988 (NAVD'88). State and local mapping systems are generally based on these systems, using either a Mercator or Lambert projection. Many localities employ, or have employed, local datums that must be correlated and reconciled. When specifying surveying or mapping work, it is necessary to indicate exactly which projection should be used.

(3) It is often appropriate, where greater accuracy is required, to develop a site-specific mapping system. Where the new structures are to be tied into existing facilities, the mapping base for the existing facilities can be extended. Often, however, it is better to modernize the system and remathematize the existing facilities as necessary.

(4) Topographic maps exist for virtually all of the United States, some of them in digital form. Depending on the age and scale of such mapping, they may be sufficient for initial planning efforts. More often than not, however, supplementary data are required, either because of inaccuracies in the available data or because of changes in land use or topography. Topographic and cultural data can be obtained from recent air photos or photos flown for the purpose, using photogrammetric techniques. Triangulation and traverses can be performed, using existing or new monuments and benchmarks, as part of the controls for photogrammetry and to verify existing mapping.

(5) Typically, reasonably detailed mapping in corridors 100 to 1,000 m (300-3,000 ft) wide are required along all contemplated alignments. This mapping should be sufficiently detailed to show natural and man-made constraints to the project. In urban areas, mapping of major utilities that may affect the project must also be procured, using utility owners' mapping and other information as available. At this time it may also be appropriate to secure property maps.

(6) Accurate topographic mapping is required to support surface geology mapping and the layout and projection of exploratory borings, whether existing or performed for the project.

(7) A computerized database, a GIS, is able to handle all of these types of information and to produce local maps and cross sections as required.

*b. Surveying and mapping tasks during design.*

(1) Mapping and profiling begun during planning must be completed during this phase. Also, all utilities must be mapped, as well as all buildings and other man-made features along the alignment. Property surveys must be completed to form the basis for securing the right-of-way.

(2) If not already available, highly accurate horizontal and vertical control surveys are required to tie down the components of the new facilities. The Global Positioning System (GPS) is helpful in providing precise references at low cost over long distances. The GPS is a satellite-based positioning system administered by the U.S. Air Force. When used in a differential mode in establishing control networks, GPS gives relative positioning accuracies as good as two ppm. GPS is also flexible, because line-of-sight is not required between points.

(3) The contract documents must contain all reference material necessary to conduct surveying control during construction. This includes generally at least the following:

- Mathematized line and grade drawings, overlain on profiles and topography from the mapping efforts. Designers will use a "working line" as a reference, usually the center or invert of the tunnel for a water tunnel, but some other defined line for transportation tunnels. All parts of the cross section along the tunnel are referenced to the working line.

- Drawings showing monuments and benchmarks to be used as primary controls. These should be verified or established for the project.

- Drawings showing existing conditions as appropriate, including all affected utilities, buildings, or other facilities.

- Interfaces with other parts of the project, as required.

- Specifications stating the accuracy requirements and the required quality control and quality assurance requirements, including required qualifications of surveyors. Where great accuracy is required, preanalysis of the surveying methodology should be required to demonstrate that sufficient accuracy can be obtained. Minimum requirements to the types and general stability of construction benchmarks and monuments may also be stated.

(4) Generally speaking, greater accuracy is required in urban areas with a great density of cultural features than in rural environments. Underground works for transportation, by their nature, require greater accuracy than most water conveyance tunnels.

(5) Benchmarks and monuments sometimes are located where they may be affected by the work or on swelling or soft ground where their stability is in doubt. Such benchmarks and monuments should be secured to a safe depth using special construction or tied back to stable points at regular intervals.

(6) Where existing structures and facilities may be affected by settlements or groundwater lowering during construction, preconstruction surveys should be conducted to establish a baseline for future effects. Such surveys should be supplemented by photographs.

*c. Construction surveying and control.*

(1) Except in rare instances, the contractor takes on all responsibilities for all surveying conducted for the construction work, including control of line and grade and layout of all facilities and structures. This permits the contractor to call on the surveyor's services exactly when needed and to schedule and control their work to avoid interferences. The owner or construction manager may

perform such work as is necessary to tie the work into adjacent existing or new construction. The owner or construction manager will also conduct verification surveys at regular intervals.

(2) The contractor's surveyor will establish temporary benchmarks and monuments as required for the work and is expected to verify the stability of these benchmarks.

(3) When a tunnel is driven from a portal, a baseline is typically established outside the portal and subsequently used as a basis for tunnel surveying. Line and grade is usually controlled by carrying a traverse through the tunnel, moving from wall to wall. This method will help compensate for surveying errors that can arise from lateral refraction problems resulting from temperature differences in the air along the tunnel walls. Rapid, high-precision survey work can be obtained using electronic levels and total-station equipment. High-precision gyrotheodolites can now provide astronomical azimuths with a standard deviation of 3 arc seconds, independent of refraction problems. This accuracy is rarely required as a standard for tunneling but is useful for verification surveys.

(4) Electromagnetic distance measuring instruments can provide accurate distance determinations between instrument and target very quickly and is the preferred method of distance measurement in tunnels.

(5) Shaft transfers have often been made using a plumb bob dampened by immersion in a bucket of water, with the vertical distance measured by a suspended tape. Two points at the shaft bottom must be established to create a baseline for tunneling. In a shaft of small diameter, the baseline thus transferred is short and therefore not accurate. In such cases, a backsight or foresight can be established by drilling a survey hole over the tail tunnel or the tunnel alignment. Such survey holes can also be used along the alignment for verification or correction in long tunnels.

(6) More modern shaft transfers are often done using an optical plummet. Vertical and horizontal shaft transfers using modern equipment, including total station, Taylor-Hobson sphere, precise level, and plummet, are accurate to depths of at least 250 m (800 ft).

(7) For a blasted tunnel, the tunnel face is marked with its center, based on laser light, and the blast layout is marked with paint marks on the face. The drill jumbo must be set accurately to ascertain parallelism of boreholes along the alignment and the proper angle of angled blastholes. An automated drill jumbo can be set up using laser light without marking the tunnel face.

(8) Modern TBMs are often equipped with semiautomated or fully automated guidance instrumentation (e.g., ZED, Leica, or DYWIDAG systems) that offers good advance rates with great precision. They require establishment of a laser line from a laser mounted on the tunnel wall. Laser beams disperse with distance and are subject to refraction from temperature variations along the tunnel wall. As a result, they must usually be reset every 250 m (800 ft) or less. For tunnels on a curve, lasers must often be reset at shorter intervals.

(9) Construction survey monuments are usually placed at a spacing of several hundred meters and at tangent points. These are sometimes made permanent marks. When placing the final, cast-in-place lining (if required), these monuments are also employed for setting the concrete forms precisely.

(10) Considering that TBMs provided with conveyor mucking systems sometimes advance at rates over 120 m/day (400 ft/day), it is evident that contractors must employ the best and fastest tools for advancing the survey controls along with the TBM in order not to slow down the advance. It is also clear that a small surveying error (or worse, a gross mistake) quickly can lead to a very costly misalignment. Thus, attention paid to the quality of the survey work and the tools used for surveying is well placed.

## 5-13. Construction Hazards and Safety Requirements

Underground construction has traditionally been considered a hazardous endeavor. Many years ago, this image was well deserved. Indeed, fatality rates during construction of classical tunnels such as the St. Gotthardt in Switzerland and the Hoosac in Massachusetts were extraordinarily high. In today's world, the frequency of accidents and the fatality rates for underground construction have approached those of other types of construction, partly because of a better understanding of causes of accidents and how to prevent them, and partly because of a greater degree of mechanization of underground works. This subsection explores common types of accidents in rock tunnels and cavern construction, their causes, and how to prevent them or to minimize their likelihood of occurrence. The potential for failures in the long term, during the operating life of tunnels, is dealt with in a later section.

*a*. *Hazards related to geologic uncertainty*. Contrary to many lay people's intuition, most tunnel accidents are not caused by rock fall or face collapse or some other geologically affected incident, but by some failure of equipment or human fallibility. Nonetheless, geologically affected failures or accidents occur, and on occasion such failures can be devastating and cause multiple fatalities. Typical accidents are discussed below.

(1) *Rock falls*. Rock falls result from inadequate support of blocks of rock that have the potential for falling or from insufficient scaling of loose blocks after a blast. Rocks can fall from the crown or the sidewalls of tunnels or from the face of a tunnel. The use of robots for installation of rock bolts or shotcrete over the muck pile after a blast greatly reduces the exposure of personnel. Rock falls also occur behind a TBM. A shielded TBM should not induce a sense of false security. Even a very small rock falling down a shaft becomes hazardous because of the high terminal velocity of the falling rock. Thus, particular attention must be paid to prevention of rock loosening around a shaft. Geologists and engineers sometimes venture out in front of the last installed ground support to map geology or to install instrumentation. More than one has been killed in this way, under a rock fall, and many have been injured.

(2) *Stress-induced failure*. Stress-induced failure occurs when in a massive or interlocking rock mass the stress induced around the underground opening exceeds the strength of the rock. Such events range in severity from delayed wedge fallouts in the crown or the sidewalls in soft rock, to popping or spalling, or violent rock bursts in hard and brittle rock.

(3) *Face or crown collapse*. This is relatively rare but can be very hazardous and costly when it occurs, as evidenced by case histories (see Box 5-1). These types of failure result either from encountering adverse conditions that were not expected and therefore not prepared for or from use of construction methods that were not suited for the adverse condition. The geological culprit is usually a zone of weakness, a fault zone with fractured and shattered rock, or soft and weathered material, often exacerbated by water inflow in large quantity or at high pressure.

(4) *Flooding or inrush of water*. Flooding or inrush of water is mostly an inconvenience, provided that adequate pumping capacity is available. The source of the water can be the interception of a pervious zone or a cavern with a substantial reservoir behind it, access to a body of water, or the breakage of a sewer or water line. In instances where the water does not naturally flow out of the tunnel, e.g., when tunneling from a shaft, adequate pumping capacity must be provided for safe evacuation. When tunneling in certain geothermally active terrains, inflow of scalding hot water can be a hazard. Large inflows of water have also occurred when tunnel construction accidentally intercepted an artesian well. When flooding brings with it large quantities of material, cohesionless sand or silt, or fault zone debris, several hundred feet of tunnel can be filled with debris or mud in a short time, causing personnel and machinery to be buried.

(4) *Gas explosions*. When gas explosions occur, they often cost a number of casualties. Examples include the San Fernando Water Tunnel in Sylmar, California, where a major methane gas explosion cost 17 lives. While recognized as a gassy tunnel, excessive amounts of gas were thought to have derived from a fault zone just ahead of the face. A Port Huron, Michigan, sewer tunnel was driven through Antrim Shale. During final lining installation, a methane explosion claimed 21 lives. More recently, a gas explosion in a tunnel in Milwaukee cost the lives of three people. The geological occurrence of methane gas is discussed in Section 3-7. Flammable and explosive gases in tunnels can (and should) be measured and monitored continuously. In some cases, automatic alarms or equipment shutdown is appropriate. Gas risks can be explored by probeholes ahead of the tunnel. Remedial actions include additional ventilation air, use of explosion-proof machinery, installation of gas-proof tunnel lining (used for the Los Angeles Metro), or predrainage of gas through advance boreholes.

(5) *Other harmful gases*. Other harmful gases may include asphyxiants as well as toxic gases (see Section 3-7):

- Nitrogen (asphyxiant) may derive from pockets in the strata.

- Carbon dioxide (asphyxiant, toxic above 10 percent) may derive from strata or dissolved in groundwater; it can result from acidic water reacting with carbonate rocks. Accumulates in depressions.

- Hydrogen sulfide (toxic) may derive from strata and groundwater, notably in volcanic terrains but also in connection with hydrocarbons. It is also present in sewer tunnels.

- Carbon monoxide (toxic) can also derive from the strata or the groundwater but is more often the result of fire.

**Box 5-1. Case History: Wilson Tunnel Collapse**

This highway tunnel on the Island of Oahu was driven with dimensions 10.4 m wide and 7.9 m high, 823 m long, through layered volcanics: basalt, ashes, clinker. Deep weathering was present on the leeward side of the range but not on the windward side. The tunnel was driven conventionally from the windward side, using full-face blasting as well as excavating tools. Ribs and lagging were used for ground support.

Driving through the relatively unweathered volcanics was uneventful. After advancing about 100 m full face into the weathered material on the leeward side, a collapse occurred some 25 m behind the face. Two weeks later, a second collapse occurred about 60 m behind the face, while the first collapse was not yet cleaned up. These two collapses did not result in casualties.

During reexcavation about 35 days after the first collapse, a third, disastrous collapse occurred, with five fatalities. Eighty meters of tunnel were buried in mud, and ground support and equipment were destroyed. Large cone-shaped depressions appeared at the ground surface.

The tunnel was eventually completed using an exploratory crown drift that acted as a drain, followed by multiple drifting. Bottom side drifts were completed first, and concrete foundations and walls placed to carry the arches constructed in crown drifts.

In this event, it appears that the contractor failed to modify his construction procedures as the ground characteristics changed drastically. Full-face excavation was not suited for this material, and the ground support was inadequate after a short period of exposure.

- Oxygen depletion can occur in soils and rocks due to oxidation of organic matter; if air is driven out of the soil into the tunnel, asphyxiation can result. Compressed-air tunneling has been known to drive oxygen-depleted air into building basements.

- Radon gas occurs mostly in igneous and metamorphic rocks, especially those that contain uranium. Radon changes into radioactive radon daughters that are harmful to the body.

Some gases, such as carbon monoxide and carbon dioxide, are heavier than air and therefore seek low points in underground openings. Workers have been asphyxiated going into shafts or wells filled with carbon dioxide. Other gases (methane) are lighter than air. Traps able to collect gases should be avoided.

(6) *Hazard reduction.* If a certain hazard exposure of a particular underground project were foreseeable, then provisions could be made to eliminate the hazard. It may be said, then, that geologic accidents or exposure to geologic hazards are the result of things unforeseen, i.e., lack of knowledge of conditions or things unforeseeable, i.e., uncertainty of behavior. These exposures also occur when danger signs are not noted, ignored, or misinterpreted. These findings form the basis for methods of hazard avoidance, as expressed in the following.

(a) Search for clues of geologic conditions that could be hazardous. Clues may be obtained from the general geologic environment—caverns in limestones, faulting and folding, deep weathering, volcanics, evidence of recent thermal action, hydrocarbons (coal, oil, or gas), unusual hydrologic regimes, hot springs, etc. Other clues should be searched for in the cultural records—records of tunneling or mining, construction difficulties of any kind, changes in hydrology, landslides, explorations for or production of oil or gas.

(b) During explorations, look for evidence of hazardous conditions. Based on the geologic environment and the initial search for clues of hazardous conditions, explorations can be focused in the most probable directions for confirmation of conditions and pinpointing hazardous locations. Tools are available to discover signs of hazards: airphoto and field mapping of geological features (faults, slides, hydrology), sampling of gases in boreholes (radon, methane, etc.), analysis of geologic structure and hydrology to extrapolate faults, discover gas traps, find anomalies of hydrostatic pressure to locate hydrologic barriers or conduits, etc.

(c) Establish plausible hazard exposure scenarios and evaluate the risks. If hazards are known with some certainty, they can be dealt with directly and in advance. For hazards of lower probability, prepare contingency plans such that the hazards will be recognized in time during construction and remedial action can be taken. Provide means for dealing with expected (and unexpected) inflows of water.

(d) Provide for discovering hazards during construction: observe, map, and interpret rock as exposed during construction; measure concentrations of gases such as methane and radon; monitor water inflow, temperature, and other relevant parameters; drill probeholes ahead of the face to intercept and locate faults and pockets of water or gas.

(e) Remedial measures could include predrainage of water-bearing rock, grouting for strengthening and impermeabilization, modification of face advance methods (shorter rounds, partial-face instead of full-face advance), ground support methods (prereinforcement, spiling or forepoling, increasing ground support close to the face, etc.), shutting down equipment depending on methane concentration, and increasing ventilation to dilute gases. Mitigation of popping and bursting rock may include shaping the opening more favorably relative to stresses and installing (yielding) rock bolts and wire fabric.

(f) Maintain rigorous vigilance, even if everything seems to go right. Perform routine observations and monitoring of the face conditions as well as the already exposed rock surfaces. Do not walk under unsupported rock unless absolutely sure of its stability. Complacence and optimism do not pay, a rock fall can happen any time.

Knowledge of and preparedness for hazardous conditions should be embodied in a written plan for hazard control and reduction, as detailed as circumstances demand. The plan should be developed during exploration and design and incorporated as a part of construction contract documents. Safety plans and procedures, as well as safety training, are required for all work; special training is required for underground workers.

*b. Hazards under human control.*

(1) As already noted, many if not most tunnel accidents are at least in part under human control or caused by human action (or inaction). The examples described below are derived from the writer's personal knowledge and experience and are not hypothetical examples.

- Person falling from height (down shaft or from elevated equipment in tunnel or cavern).
- Person falling on the level (stumbling over equipment or debris left on floor, slipping on slick surfaces, exacerbated by often cramped conditions, limited space for movement, and poor lighting).
- Material falling from height (down the shaft, from equipment or vehicles, or from stacks or piles of material), including ice formed from seepage water.
- Interference with special tunneling equipment (person crushed by concrete lining segment erector or rock bolter, mangled in conveyor belt, or other moving piece of equipment—sometimes due to equipment malfunction, more often due to human error).
- Overstress of rock bolt or dowel or failure of anchorage during testing or installation, causing sudden failure of metal and a projectile-like release of metal (do not stand in the line of bolts or dowels tested).
- Moving-vehicle accidents (inspector run down by muck train or other vehicle, loco operator facing the wrong way hit by casing protruding down from the tunnel crown).
- Rock falls due to failure to recognize need for reinforcement.
- Electric accidents, electrocution (electrician failing to secure circuits before working on equipment, faults due to moisture entering electric equipment).
- Blasting accidents (flying rock, unexploded charges in muck pile, premature initiation, which could occur due to stray currents or radio activity, if using electric detonation).
- Fire and explosion other than from natural gas (electric fault as initiator, fumes from burning plastic, electric insulation, and other materials, burning of timber can result in loss of ground support, generation of carbon monoxide and other poisonous or asphyxiating gases).
- Atmospheric pollution due to equipment exhaust, explosives fumes, or dust generated from

- explosion, equipment movement, muck transport by cars or conveyor, dry or wet shotcrete application or TBM operation. Certain grouts have been known to release fumes during curing. Remedial measures: adherence to ventilation requirements, face masks.

- Heat exhaustion due to high temperature and humidity (preventable by adherence to regulations regarding thermal exposure).

- Excessive noise from drilling equipment, ventilator, or from blasting (ear plugs required).

(2) It is apparent that most of these types of accident or risk exposure could happen in many locations outside the tunnel environment. In fact, most of them are typical construction accidents. If they happen more commonly in the underground environment, it is for several reasons:

- Tunnels often provide very limited space for work and for people to move; thus people move slower and have a harder time getting out of the way of hazards.

- Poor lighting and limited visibility in the tunnel are other contributing factors.

- Often inadequate instruction and training of personnel in the detailed mechanics of tunneling make personnel inattentive to hazards and put them in the wrong place at the wrong time.

- Carelessness and inattention to safety requirements on the part of workers or supervisory personnel; unauthorized action on part of worker.

- Equipment failure, sometimes due to inadequate inspection and maintenance.

Prevention of accidents in tunnels and other underground works requires education and training of all personnel and rigorous and disciplined enforcement of safety rules and regulations during construction.

*c*. *Safety regulations and safety plans*.

(1) Safety of underground works other than mines is regulated by OSHA—the Occupational Safety and Health Act, 29CFR1926. Numerous other regulations govern various aspects of underground safety:

- The Internal Revenue Service 26CFR Part 181, Commerce in Explosives.

- 27CFR Part 55, administered by the Bureau of Alcohol, Tobacco and Firearms (both regulate manufacturing, trading, and storage as well as safekeeping of explosives).

- Department of Transportation 49CFR Part 173 and other Parts (regulate transportation of explosives).

- For DOD work, DOD 6055.9 - STD, Ammunition and Explosives Safety Standards, and DOD 4145.26 M, DOD Contractors Safety Manual for Ammunition and Explosives apply.

- The National Electric Code applies to all temporary and permanent electrical installations.

- MSHA - Mine Safety and Health Act, 30CFR Part 57 among other things defines and lists vehicles permissible underground.

(2) Among other documents that apply, American Congress of Government Industrial Hygienists' (ACGIH) Threshold Limit Values for Chemical Substances and Physical Agents in the Workroom Environment (1973) is important for ventilation of the underground. U.S. Environmental Protection Agency (EPA) regulations apply to handling and disposal of hazardous materials and contaminants.

(3) While, strictly speaking, the USACE is empowered to enforce its safety regulations on USACE projects, it is the practice to permit OSHA inspection and enforcement privileges. Where local regulations exist and are more stringent than OSHA, they are usually made to apply. An example of regulations exceeding OSHA in strictness is the State of California Tunnel Safety Orders.

(4) Contractors are obliged to follow all applicable Federal, state, and local laws and regulations and are generally responsible for safety on the job. Nonetheless, it is appropriate in the contract documents to reference the most important laws and regulations. It is also proper to require of the contractor certain standards and measures appropriate to the conditions and hazards of the project and for the USACE's resident engineering staff to enforce these standards and measures.

(5) For complicated or particularly hazardous projects, it is common to require the preparation of a Safety Analysis Report, in which all construction procedures are analyzed by the contractor, broken down to detailed subcomponents. The report also identifies all hazards, such that preventive and mitigating procedures can be developed and emergency measures prepared.

(6) For all projects, the contractor is required to prepare a full Safety Plan, subject to review and approval by the resident engineer, who will employ this plan for enforcement purposes. The act of preparing a project-specific Safety Analysis Report and Safety Plan, rather than using a standard or generic plan, will alert the contractor and the resident engineer to particular hazards that might not be covered by a standard plan, and will heighten the level of attention to safety provisions. Components of a typical Safety Plan may include the following types of items and other items as appropriate:

- Policy Statement: Elimination of accidents, no lost time due to accidents, safety takes precedence.

- References: Applicable laws and regulations.

- Responsibilities: Chains of command, administration and organization of safety program, authorizations required before commencing work, enforcement.

- Indoctrination and Training: Required training program for all, separate program for underground workers, required weekly toolbox safety meetings, requirements for posting information, etc.

- General Safety and Health Procedures: Housekeeping, material handling and storage, personal protective equipment, dealing with wall and floor openings, scaffolds, ladders, welding, flame cutting, electrical equipment, lock-out or tag-out procedures, motor vehicles, heavy equipment, small tools, concrete forms, steel erection, cranes and hoisting, work platforms, fire prevention and protection, sanitation, illumination, confined space entry, etc.

- Industrial Hygiene: Respiratory protection, noise, hazardous materials, submittal of Material Safety Data Sheets (MSDSs) and lists of hazardous chemicals present, hazards communication.

- Emergency Procedures: Detailed procedures for all types of emergencies, medical, fire, chemical spill, property damage, bomb threat, severe weather.

- Incident Investigation, Reporting, Record Keeping.

- Policy for Substance Abuse.

- Security Provisions.

(7) Additional provisions applicable to underground works include safety of hoisting, blasting safety, use of CO and $CO_2$ breathers (self-rescuers), which convert these gases to oxygen, access and egress control including emergency egress, safety inspection of exposed ground, storage of fuel underground, communications underground, monitoring of gases and dust in the tunnel, lighting and ventilation in the tunnel, and requirements to establish trained rescue teams.

(8) Depending on the number of people in the contractor's work force and the number of shifts worked, the contractor may be required to employ one or two persons who are fully dedicated safety officers. Likewise, one or more safety officers may also be required on the resident engineer's staff. Safety engineers are authorized to stop the work if a hazardous condition is discovered that requires work stoppage for correction. With proper cooperation and timely action, such work stoppages usually do not occur.

(9) Construction safety is serious business and must command the fullest attention of management personnel on all sides. An effective safety program relies on the following:

- Planning to avoid hazards.

- Detection of potential hazards.

- Timely correction of hazards.

- Dedication to the protection of the public and the worker.

- Active participation of all persons on the job.

- Dedicated safety staff.

## 5-14. Environmental Considerations and Effects

Many laws, rules, and regulations apply to underground construction. The National Environmental Policy Act, the

Clean Water Act, the Rivers and Harbors Act, the Endangered Species Act, and various regulations pertaining to historic and cultural resources are the major requirements that apply primarily to preconstruction phases. Regulatory programs that apply to construction include the following:

- Resource Conservation and Recovery Act (RCRA).

- Comprehensive Environmental Response, Compensation and Liability Act (CERCLA), also known as the Super Fund Act, including SARA Title III

- National Pollutant Discharge Elimination System (NPDES) permit program that is part of the Clean Water Act.

Satisfying the requirements imposed by these laws and regulations including associated permits are the focus of other documents and are not addressed in this manual. Accommodating environmental and permit requirements during construction involve little incremental cost or schedule disruption if the requirements are effectively addressed in planning, design, and contract documents. Early preconstruction work typically includes preparation of an Environmental Impact Statement (EIS). Design and construction constraints embodied in the EIS must be adhered to during design and construction.

*a. Effects of settlements and ground movements.*

(1) Ground movements and settlements occur either as a result of elastic or inelastic relaxation of the ground when excavation relieves in situ pressures or as a result of groundwater lowering. Lowering the groundwater table can result in compaction or consolidation of loose or soft overburden. Removal of fines by seepage water or via dewatering wells can also result in settlements. Gross instability and collapse of tunnel face (or shaft bottom) also cause ground surface depressions.

(2) Tunnels and shafts in rock, when properly stabilized, usually do not result in measurable ground settlements. On the other hand, ground movement control is a major issue for tunnels and excavations in soil in urban areas, especially if below the groundwater table.

(3) When damaging settlements are deemed possible for a rock shaft or tunnel project (e.g., shaft through overburden, effect of dewatering), the following provisions should be taken:

- Preconstruction surveys with photos or video, documenting existing conditions.

- Contract requirements to limit or eliminate effects that can cause settlements.

- Monitoring of construction performance (measurements of ground motions, settlements of buildings, groundwater level, etc.).

- Provisions to pay for damage, if any (cost sometimes to be borne by the contractor).

(4) In general, contractual provisions should be devised that will encourage the contractor to conduct his work with a minimum of ground motions.

*b. Groundwater control and disposal.*

(1) Groundwater levels should be maintained during construction, if practicable, to avoid a number of risks including unexpected ground settlement, entrainment of pollutants from underground tanks or other sources, affecting surface water systems, and water quality concerns associated with disposal. If shafts are required for tunnel access, methods of shaft sinking should be adopted that do not require aggressive pumping to create a cone of depression prior to installation of the lining.

(2) In many cases, excessive infiltration of groundwater into tunnels and shafts during or after construction is unacceptable because wells owned and operated by private persons or public agencies may be seriously affected by lowering of the groundwater. Concern for the natural environment, including existing vegetation, springs, and creeks, can require tight control of water infiltration both during construction and operations. Monitoring of the surface hydrology as well as observation wells is often required to ascertain effects of tunneling and show compliance with performance restrictions. If unacceptable effects are found, remedial action may be required.

(3) Effective management of tunnel seepage includes discharge to onsite settling ponds or tanks of sufficient capacity to reduce suspended solids to acceptable levels before discharging tunnel seepage into a storm water system or surface stream. The water management system should also have a means of detecting and removing petroleum hydrocarbons prior to discharge. This can be accomplished through an oil-water separator or passing the discharge through oil-sorbent material in combination with a settling basin or pond.

(4) Widely accepted standards for hydrocarbon concentrations in discharged water are “no visible sheen” and no more than 15 parts per million (ppm). The acceptable

pH range for discharged water often is between 6.0 and 9.0, although some states or localities may have narrower limits. Standards or policies established during the design should be incorporated in the contract requirements so that compliance costs will be reflected in bids.

(5) Conflicts with agency staff and landowners will be minimized if contractors clean up leaks and spills in the tunnel, conduct grouting and shotcrete activities so as to prevent highly alkaline water from leaving the site, and have emergency equipment and materials on hand to effectively manage water that may become contaminated by a construction emergency.

(6) Frequent, systematic site inspections to evaluate construction practices are effective in documenting conditions and in identifying corrective action that must be taken. Corrective actions can also be tracked and closed out after being implemented. Documentation includes photographs and water quality data from onsite ponds and discharge.

(7) Leakage from underground tanks and pipelines, leachate from landfills, or contamination from illegal dumping or surface pits are a few of the conditions that may be encountered during tunneling. Preconstruction surveys can provide an indication if current or past land uses are likely to have contaminated areas where the tunnel will be constructed. In such cases, the designer should anticipate possible adverse effects on tunnel linings as well as measures for proper management and disposal. In the extreme, aligning the tunnel to avoid such areas may be the most cost-effective solution. Avoidance also limits the potential long-term liability that is associated with handling and disposing of contaminated solids and liquid wastes.

(8) Unexpected contamination can occur where underground fuel tanks have been in use for many years. Overfilling and leaks can result in high concentrations of gasoline and fuel oil, which present a hazard to work crews as well as high costs for disposal. Other potential sources of contamination include commercial cleaning shops and abandoned industrial facilities.

(9) The environmental hazard and liability are often minimized by contracting, in advance of construction, with a firm that will provide emergency response. This would include services to contain contamination, test water and soils to determine the types and concentrations of contaminants, provide advice on possible contamination sources, and advise and assist in proper disposal. Alternatively, contamination could be removed before tunneling.

(10) On occasion, a water supply tunnel will traverse a region of brackish groundwater or brine or water containing other unacceptable chemicals. Here, a nominally watertight lining must usually be provided to minimize infiltration. In the case of sewer tunnels, exfiltration can contaminate surrounding aquifers. Sewer pipes and tunnels must usually meet water tightness requirements laid down by local authorities.

*c. Spoil management.*

(1) Disposal of material removed from tunnels and shafts is often the source of considerable discussion during the environmental planning phase.

(2) In rural areas, tunnel muck can often be disposed of onsite without adversely affecting surface or ground water. In urban areas, it may have to be transported to locations well removed from the point of generation. Except for special circumstances, tunnel muck in the urban environment is usually disposed of by the contractor, who is obliged to follow applicable regulations.

(3) Total petroleum hydrocarbon concentrations in soil, muck, or sediment can restrict management and disposal options. A widely accepted criterion for total petroleum hydrocarbon concentration is 100 ppm. Muck up to this concentration can be disposed of onsite, whereas muck with higher concentrations requires special disposal. The requirements for a specific project location should be determined during the design and included in the contract documents. The costs for managing muck that exceeds criteria are typically high and can be an inducement for contractors to carefully handle fuels and oils. It is often thought that tunnel muck produced by a TBM is useful as concrete aggregate. TBM muck, however, consists of elongated and sharp-edged pieces of rock, unsuitable for concrete aggregate. Recrushing generally does not help. TBM muck, however, is useful as road fill.

(4) The size and shape of spoil piles is frequently an issue once the location has been determined. Maximum pile height and sideslope grade, desirable configurations or shapes, and permanent ground cover would be determined based on the specific of each project.

(5) RCRA, CERCLA, NPDES, and state rules and regulations can involve special management techniques. Waste water and spoil that has naturally high heavy metal content, has high levels of radioactive isotopes, or is contaminated by some action or facility owned by others could produce harmful leachate. The potential for these to

occur depends on the location and nature of the project. Construction monitoring to detect, characterize, and properly manage the disposal of excess material should be conducted to document that spoil is being properly handled.

*d. Waste Waters*. Equipment and construction may generate "process" waste waters that require Federal or state permits to discharge into surface waters. Federal and state regulations may require a permit to discharge TBM cooling water, wash water from scrubbers, waste from onsite treatment processes, pipe flushes and disinfectants, or other nonstorm waters. Regulatory requirements are determined from the particular type of nonstorm water discharged, even if it meets the highest standard of quality. The contract documents should indicate which waters cannot be discharged into surface drainage if permits cannot be acquired prior to contract award.

*e. Control of fugitive dust*.

(1) The 24-hr and annual National Ambient Air Quality Standards (NAAQS) established for dust particles 10 µm are maximum 150 µg/m$^3$ and 50 µg/m$^3$ of air, respectively. Such particles tend to become trapped in lungs and pose a long-term hazard. Larger particles are not always regulated by a quantitative standard, but can result in regulatory action if there are complaints. Stringent dust control standards may apply to construction fugitive dust emissions for projects located in air sheds that do not meet the NAAQS for particulates.

(2) Confining dust to a construction site is difficult if the site is small, the rock tends to produce a large percentage of fines, and the contractor's muck handling method involves a number of transfers, or there is heavy traffic on unpaved roads. Raising the moisture content of muck with water in combination with shrouds or other devices is an effective measure to confine dust in the work area. This frequently involves situating spray nozzles at vent outlets, along conveyor transfer points, on stackers, and on temporary muck piles that will be loaded and transported to the disposal area. Paved construction roads are also an effective dust control measure. Establishing a criterion for "no visible dust" outside the construction boundary and leaving the means and methods to contractors may not result in acceptable dust control.

*f. Storm water runoff and erosion control*.

(1) A general NPDES permit to discharge storm water from construction sites larger than 5 acres was published by EPA in the Federal Register, September 9, 1992. The permit requires EPA to be notified when construction is started and completed but requires no other routine filings. A storm water pollution prevention plan (SWPPP), various certifications, and periodic site inspections are to be maintained at the construction site. The plan must be site specific and address techniques to divert overland flow around disturbed areas, stabilize slopes to prevent erosion, control runoff from disturbed areas so that sediment is trapped, prevent mud from being tracked onto public roads, and properly store and handle fuel, construction chemicals, and wastes.

(2) The SWPPP must satisfy standards contained in the regulations. Contractually, this could be accomplished by setting a performance standard or by developing a detailed plan that the contractor must implement. The former approach enables contractors to apply their expertise and knowledge of the area and relieves designers of predicting a contractor's requirements for temporary facilities. It does, however, put the owner at risk if the contractor does a poor job of planning or executing the plan.

(3) The latter approach gives the owner much more control over compliance. The procurement documents would contain the plan and a copy of the filed Notice of Intent, as well as a partially completed notice of termination, which the contractor would complete and file at the end of the job. The contractor could make changes in the storm water plan, but only after proposing them in a form that could be incorporated into the plan and receiving written approval from the owner.

*g. Noise and vibration*.

(1) Incorporated urban areas typically have noise and vibration ordinances that may apply to tunneling. These would be satisfied by surrounding noise sources in acoustical enclosures, erecting sound walls, limiting noise-generation activities to certain times of the day, or by using equipment designed to achieve reduced noise levels.

(2) Acceptable construction noise levels at a sensitive receptor (e.g., dwelling, hospital, park) may be established for day and night by state or local agencies. Some degree of noise monitoring prior to and during construction is advisable. An integrating precision sound level meter that provides maximum, minimum, and equivalent (average) noise outputs is appropriate. A typical day and night noise level limit for rural areas is 55 dBA and 45 dBA, respectively. For residential areas in cities, acceptable noise levels would typically range between 65 dBA and 75 dBA, with higher levels for commercial areas.

(3) Vibration and air-blast noise are usually associated with blasting, an activity that is readily controlled to achieve applicable standards. Monitoring and control of blasting vibrations are discussed in Section 5-1-e.

*h. Contingency planning.*

(1) Underground construction can encounter unexpected conditions and involve incidents that can release pollutants into the environment. Developing strategies to accommodate the types of events that could result in polluting water and soil is an effective method to reduce impacts and liability. Examples of pollution-causing incidents include a massive loss of hydraulic fluid in the tunnel, large inflow of groundwater, rupture of diesel fuel tank on the surface, vehicle accident involving diesel spill, fire, and the release of hazardous construction chemicals.

(2) Advance planning strategies include proper storage of fuels and chemicals, secondary containment, good housekeeping, training for all persons in corrective actions during incidents, bolstered by periodic discussion in tool box sessions, stockpiling response kits and containers to initiate proper cleanup, and having a contract in place with qualified emergency response personnel.

(3) The requirements contained in 40 CFR 112, which requires a spill prevention, control, and countermeasures plan if certain oil storage limits are exceeded, provides a good model on which to start contingency planning.

## 5-15. Contracting Practices

A principal goal in preparing contract documents is to achieve a contract that will yield a fair price for the work performed, acceptable quality of the work, and a minimum of disputes. A number of different contract provisions are employed to achieve these goals. Several of these clauses serve to minimize the need for bidders to include large contingencies in their bid to make up for the uncertainty often associated with underground works. The USACE employs a large number of standard provisions and clauses in the preparation of contract documents. Many of these can be used for underground works as they are, but a number of them require modifications to make them apply to the particular working conditions and project requirements of underground works. Each clause contemplated for use should be read carefully and modified as required. As an example, concrete placement for a final lining is very different from concrete placement for surface structures. Specifications for initial ground support, as well as for tunnel and shaft excavation, must usually be tailored to conditions for the particular tunnel.

*a. Clauses.* A number of clauses are of particular use in underground works; these are discussed briefly in the following.

(1) *Differing site conditions.* The differing site conditions clause is now a standard in most contracts, including those funded by Federal moneys. The clause provides that the contractor is entitled to additional reimbursement if conditions (geologic or other) differ from what is represented in the contract documents and if these conditions cause the contractor to expend additional time and money.

(2) *Full disclosure of available subsurface information.* All available factual subsurface information should be fully disclosed to bidders, without disclaimers. This is usually achieved by making all geotechnical data reports available to the bidders. In addition, the designer's assessment as to how the subsurface conditions affected the design and the designer's interpretation of construction conditions are usually presented in a GDSR. This report is usually made a part of the contract documents. This report should carefully define what the contractor can assume for his bid, which risks are to be borne by the owner and which by the contractor, and what will be the basis for any differing site conditions claims. The use of the GDSR as a baseline document is not at this time a standard practice for USACE projects.

(3) *Contract variations in price.*

(a) When a construction contract is relatively small and the work is well defined with little chance of design changes, and when the geology is well defined, a lump sum type of contract is often appropriate. Most often, however, underground construction contracts are better served by another type of contract in which certain well-defined parts of the work are paid for in individual lump sums, while other parts are paid for on a unit price basis. This permits equitable payment for portions of the work where quantities are uncertain.

(b) As an example, the required initial ground support in a rock tunnel is not known with certainty until conditions are exposed in the tunnel. It is common practice to show three or more different ground support schemes or methods, suitable for different rock quality as exposed. For each scheme, the contractor bids a unit price per foot of tunnel. The designer provides an estimate of how much of each type of ground support will be needed; this estimate provides the basis for the contractor's bid. The contractor will then be paid according to the actual footage of tunnel where each different ground support scheme is required.

(c) Other construction items that may be suited for unit pricing include the following as examples:

- Probehole drilling, per meter (foot).
- Preventive or remedial grouting, per meter (foot) of grout hole, per hookup and per quantity of grout injected.
- Supplementary payments if estimated water inflow is exceeded, possibly on a graduated scale.
- Different payment for excavation of different rock (soil) types if excavation efforts are expected to be significantly different and quantities are unknown.
- Payment for stopping TBM advance (hourly rate) if necessary to perform probehole drilling or grouting or to deal with excessive groundwater inflow or other defined inclement.

(d) When preparing a bid schedule with variable bid items, it is wise to watch for opportunities where the bidder could unbalance the bid by placing excessive unit prices on items with small quantities. Each quantity should be large enough to affect the bid total. In some cases, unit prices are "upset" at a maximum permitted price to avoid unbalancing.

(e) There is usually a standard clause providing for adjustment to unit prices if changes in quantities exceed a certain amount, usually 15 or 20 percent. Depending on the certainty with which conditions are known, some or all of the unit prices discussed here may be excluded from this clause.

*b. Other contracting techniques*.

(1) *Dispute Review Board*.

(a) Legal pursuit of disputes arising from contractor claims are expensive, tedious, and time-consuming. Disputes also bring about adversary relations between contractor and owner during construction. Dispute Review Boards (DRBs) go a long way toward minimizing or eliminating disputes by fostering an atmosphere of open disclosure and rapid resolution during construction, when the basis for any claims is still fresh in memory. The use of DRBs is extensively described in ASCE (1994).

(b) The typical DRB consists of three members—one selected by the contractor, one by the owner, and one by the first two members, all subject to approval by both parties and all experienced in the type(s) of work at hand and in interpreting and understanding the written word of the contract. The DRB members must have no vested interest in the project or the parties to the construction contract other than their employment as DRB members. The DRB usually meets every 3 months to familiarize themselves with the project activities. Claims between the contractor and the owner that have not been resolved will be brought before the DRB, who will render a finding of entitlement and, if requested, a finding of quantum (dollars, time). These findings are recommendations only and must be agreed to by both parties. The contractor will still have legal recourse, but the findings of the DRB are admissible as evidence in court.

(c) Because the DRB members have no monetary interest in the matter (other than their DRB membership), and because they are usually seasoned and respected members of the profession, their findings are almost always accepted by the parties, and the dispute is resolved in short order, while the matter is fresh and before it can damage relations on the job site.

(2) *Escrow of bid documents*. DRBs are usually recommended in conjunction with the use of escrowing of bid documents (ASCE 1994). A copy of the contractor's documentation for the basis of the bid, including all assumptions made in calculation of prices, is taken into escrow shortly after the bid. At the time of escrow, the documents are examined only for completeness. The documents can be made available to the parties of the contract and the DRB if all parties agree. By examining the original basis for the bid, it is often found easier to settle on monetary awards for contract changes and differing site conditions.

(3) *Partnering and shared risk*.

(a) The USACE introduced the concept of partnering in 1989. It includes a written agreement to address all issues as partners rather than as adversaries. Contracting issues involving risk sharing and indemnification may be discussed within the partnering agreement. This requires both training and indoctrination of the people involved. Partnering also includes at least the following components:

- A starting, professionally guided workshop of 1 or 2 day's duration, where the emphasis is on mutual understanding and appreciation and development of commitments to work together with team spirit.

- Continuing periodic partnership meetings, usually addressing job problems but structured to approach them as partners rather than antagonists; a professional facilitator usually leads these meetings.

(b) Experience with partnering has been good, and it is felt that this device has reduced the number of disputes that arrive in front of the DRB. Partnering will not resolve honest differences of opinion or interpretation but will probably make them easier to resolve.

(4) *Prequalification of contractors*. For large and complex projects requiring contractors with special expertise, it is common to prequalify contractors for bidding. For USACE projects this is rarely done. Some time before contract documents are released for bidding, an invitation is published for contractors to review project information and submit qualifications in accordance with specific formats and requirements prepared for the project. Only those qualifying financially as well as technically will be permitted to submit bids on the contract. Prequalification can apply to the contracting firm's experience and track record, qualifications of proposed personnel, and financial track records.

## 5-16. Practical Considerations for the Planning of Tunnel Projects

For many tunnels, size, line, and grade are firmly determined by functional requirements. This is true of most traffic tunnels as well as gravity sewer tunnels. For other types of tunnels, these parameters can be selected within certain bounds. A summary is presented below of a few practical hints for the planning of economical tunnels.

*a*. *Size or diameter of tunnel*. Hard-rock TBMs have been built to sizes over 10 m in diameter (33 ft); span widths for blasted openings are restricted only by rock quality and rock cover. For rapid and economical tunneling of relatively long tunnels, a diameter of about 4.5 m (15 ft) or larger (3.5-m (11.5-ft) width for horseshoe shape) is convenient. This tunnel size permits the installation of a California switch to accommodate a 1.07-m (42-in.) gage rail, which allows passing of reasonably sized train cars. Smallest tunnel diameter or width conveniently driven by TBM or blasting is about 2.1-2.4 m (7-8 ft). Pilot or exploratory tunnels are usually driven at a size of 2.4-3 m (8-10 ft), depending on length. Smaller tunnels can be driven by microtunneling methods.

*b*. *Shaft sizes*. Shafts excavated by blasting should be at least 3-3.5 m (10-12 ft) in size; the maximum size is not limited by the method of excavation. Blind drilling with reaming using triple kelly is limited to about 7.5 m (25 ft) at 80 m (270 ft) depth. Blind drilling using reverse circulation can produce shafts to a diameter of more than 6 m (20 ft), depending on depth and rock hardness. The maximum depth achieved using blind drilling through hard rock is in excess of 1,000 m (3,300 ft), with a drilled diameter of about 3 m, and finished diameter of the steel casing of 1.8 m (6 ft). Raise drilling is currently limited to about 6 m (20 ft) in diameter, depending on rock strength and hardness.

*c*. *Grade or inclination of tunnel*. With rail transport in the tunnel, a grade of 2 percent is normal, and 3 percent is usually considered the maximum grade. Higher grades—up to more than 12 percent—can be used with cable hoisting gear or similar equipment. Rail transport usually occurs at a maximum velocity of 15 mph. Rail transport has limited flexibility but is economical compared with rubber-tired transport for longer (> about 1.6 km (1 mi)) tunnels. Rubber-tired equipment can conveniently negotiate a 10-percent grade, but up to 25 percent is possible. The usual maximum speed is about 25 mph. For conveyor belts, a grade of 17 percent is a good maximum, though 20 percent can be accommodated with muck that does not roll down the belt easily. Depending on belt width, the maximum particle size is 0.3-0.45 m (12-18 in.). Most belts run straight, but some modern belts can negotiate large-radius curves. Pipelines using hydraulic or pneumatic systems can be used at any grade but are rarely used. Usually, shafts shallower than 30 m (100 ft) employ cranes for hoisting; a headframe is used for deeper shafts. Vertical conveyors are used for muck removal through shafts to depths greater than 120 m (400 ft).

*d*. *TBM versus blasting excavation of tunnels*. No hard and fast rules apply on the selection of excavation methods for tunneling. The economy of TBM versus other mechanical excavation versus blasting depends on tunnel length, size, rock type, major rock weaknesses such as shear zones, schedule requirements, and numerous other factors. Cost and schedule estimates are often required to determine the most feasible method. On occasion, it is appropriate to permit either of these methods and provide design details for both or all. From a recent survey of USACE tunnels, all tunnels greater in length than 1,200 m (4,000 ft) were driven with TBM, and all under that length were driven using blasting techniques. Tunnels driven by USACE also include roadheader-driven tunnels in Kentucky, West Virginia, and New Mexico, all about 600 m (2,000 ft) long.

*e*. *Staging area*. Where space is available, the typical staging area for a tunnel or shaft project can usually be fitted into an area of about 90 by 150 m (300 by 500 ft). An area of this size can be used for space-planning purposes. If space is restricted, for example in an urban area, there are many ways to reduce the work area requirements, and many urban sites have been restricted to areas of 30 by 60 m (100 by 200 ft) or less. Such constraints cause contractor inconvenience, delays, and additional costs. If contaminated drainage water must be dealt with, the water treatment plant and siltation basin must also be considered in the estimate of work area requirements.